THE DESIGN, CONSTRUCTION, AND USE OF A STATIC PENETROMETER IN MICACEOUS SILTS OF THE SOUTHERN PIEDMONT REGION

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SUMMARY

These investigations were undertaken to study the practicality of using a static penetrometer in foundation investigations in the soils of the Southern Piedmont Region. The information obtained was used in a study of the bearing capacity of piling in these soils.

The first step of these studies was to design and construct a static cone penetrometer. Next, static penetration tests were performed at selected sites, accompanied by auger borings with undisturbed sampling and Standard Penetration Tests. The undisturbed samples were used to determine from laboratory tests the soil properties for theoretical study and a comparison of measured penetration resistances.

It has been determined by these studies that the measured point resistance of the penetrometer compares very well with computed values under the following conditions:

1. Prandtl-Reissner (shallow) bearing capacity factors are used.

2. All shape factors are given a value of unity.

3. Surface term is neglected in the bearing capacity equation.

It is also noted that point resistances were significant only in hardest materials.

There also appears to be a relation of the penetrometer resistance to the Standard Penetration Test blow count, "N," of:

$$q_o = 33.3N \quad \text{(psi)}$$

or

$$q_o = 2.4N \quad (\text{T/ft.}^2)$$

This work also indicates that the effective adhesion on a smooth steel pile shaft is approximately 65% of undrained shear strength determined by triaxial tests if $K = 1$ and $\delta = \phi$ is assumed.

Equations for the bearing capacity of piling, based on static penetration tests and the Standard Penetration Test are also presented.

CHAPTER I

INTRODUCTION

For many years attempts have been made to determine subsurface soil properties by driving or pushing rods and pipes into the ground. This study encompasses the development of an efficient cone penetrometer for the measurement of point and skin resistances to penetration.

The static penetration test has the distinct advantage of being a continuous load test, carried to failure, throughout the range of a proposed foundation. This type of test is useful, in addition to determining the over-all resistances, in locating any soft pockets or hard layers in the subsoil profile. The static penetrometer, however, does suffer from the disadvantage that substantial reactions are required in the denser soils; thus its usefulness is limited to the less dense soils.

As the penetration tests are performed, the soil around the penetrometer point and shaft fails, as would the soil adjacent to a pile loaded to failure in the soil. These tests should therefore give a better idea as to the ultimate bearing capacity of the micaceous silts under discussion.

The soils considered in this investigation are the residual soils of the Southern Piedmont region. These soils are formed by the chemical weathering of the igneous and metamorphic parent rocks. The underlying rocks consist mainly of granites, gneisses, and mica schists.

The soils in this area are predominantly silts by grain size; however, they vary from silty clays to silty sands, depending upon the parent rock and the degree of weathering. All of these soils contain a high percentage of mica.

A typical soil profile in this area from the surface down to bedrock would be: one to ten feet of red or brown silty clay, overlying ten to fifty feet of loose micaceous silts, then a layer of weathered rock overlying the intact parent rock.

The soils important to this study lie in the intermediate range and are classed as the micaceous silts.

CHAPTER II

HISTORY OF STATIC PENETROMETERS

Probably the earliest full-scale use of static penetrometers in soil exploration was by the Swedish Railway in 1917. This investigation consisted of measuring the static resistance to an auger-shaped point while being pushed into the ground. The information thus obtained was correlated with soil properties and then used in connection with design of railroad embankments.

In 1927, the Danish Railway System began using a static penetrometer with a point in the shape of a truncated pyramid. The resistance of the shaft was measured and was used in predetermining pile lengths.

The first real user and developer of a static penetrometer consisting of separate point and shaft was the Delft Soil Mechanics Laboratory in Holland in 1935. This encased penetrometer with the 60° conical point has since been known as a Dutch Cone.

Since 1935 the Delft Laboratory, in connection with the Department of Public Works of Holland as well as the National Geotechnical Institute of Belgium, performed thousands of Dutch cone soundings throughout Holland and Belgium. This procedure consisted of pushing the components separately for a distance of 20 inches. The results were then used in the determination of the bearing capacity of piles and shallow foundations. The penetrometer used has an end area of 10 sq. centimeters and is pushed at a rate of approximately one centimeter per second.

The first work with the Dutch cone at the Delft Laboratory was under the direction of P. Barentsen and this was later extended by T. K. Huizinga.

E. E. De Beer (5) has made comprehensive studies in the use of static penetrometers for determination of soil properties for use in pile foundation design. In analyzing the results of deep cone penetration tests De Beer makes the following assumption:

(1) The material beneath the point of the cone is incompressible.

(2) The lateral confinement of the soil is proportional to the overburden.

(3) Undrained triaxial shear tests are applicable for determining the strength parameters of clayey and silty soils.

De Beer further suggests that skin resistance on the penetrometer be determined by subtracting the point resistance from the total casing resistance. This will reduce the chance of erroneous readings due to friction within the penetrometer.

Theoretical and experimental studies have revealed the fact that the width of the pile or penetrometer does not appreciably affect the unit end bearing capacity. Thus the bearing capacity equation is reduced to

$$q = cN_c + q'N_q$$

The magnitude of the usual third term in the bearing capacity equation, $1/2\gamma bN_\gamma$, is very insignificant when compared with the entire end bearing capacity. It is, therefore, omitted with a resulting negligible error.

Studies of static, deep cone penetrometers in clays by C. Van Der Veen (22) show that the unit point resistance of a pile is very close to that of the unit point resistance of the penetrometer. Van Der Veen states that the resistances used for the unit cone resistances should be the average readings of the penetrometer over the ranges of one pile diameter below the proposed tip elevation and 3.75 pile diameters above this elevation. With these calculations, he also suggests the use of a factor of safety equal to 3.0.

Work by L. Bogdanovic (3) indicates that in alluvial deposits of sands and clays it is possible to sufficiently estimate the skin resistance on a pile by direct extrapolation from penetrometer resistances. His work does not include any correction for the change in material from the steel penetrometer to a timber or concrete pile.

Bogdanovic also advances the use of average values in the determination of point resistances of the penetrometer and piles. The average values, he states, should be from the range of 4 pile diameters below this elevations.

G. G. Meyerhof (15) has performed extensive investigations in the use of penetration tests for pile foundation design. This work has resulted in the following correlations between the Standard Penetration Test blow count, static penetration tests, and pile bearing capacity.

1. Relation between static penetrometer resistance and pile bearing capacity:

$$Q_{ult.\ of\ pile} = q_o A_p + 2 f_s A_s \quad \text{(T)}$$

2. Relation of penetrometer point resistance and Standard Penetration Test blow count "N."

$$q_o = 4\overline{N} \quad \text{(T/sq. ft.)}$$

3. Relation of penetrometer skin resistance and Standard Penetration Test blow count "N."

$$f_s = \frac{\overline{N}}{100} \quad \text{(T/sq. ft.)}$$

In the above equations, the following notations are used:

A_p - area of pile point (sq. ft.)

A_s - area of pile shaft (surface) (sq. ft.)

q_o - static penetrometer point resistance (T/sq.ft.)

f_s - static penetrometer skin resistance (T/sq. ft.)

N - Standard Penetration Test Blow Count

$\overline{N}$ - equivalent blow count

$$\overline{N} = N \qquad \text{if } N < 15$$

$$\overline{N} = N + \tfrac{1}{2}(N-15) \qquad \text{if } N > 15$$

These equations have been developed by Meyerhof for use in cohesionless soils. Therefore he uses the point resistances over the entire length as an indication of the potential earth pressure against the side of the pile. This eliminates measuring penetrometer skin resistances and the determination of active cohesion or adhesion.

Meyerhof also states that there is a similar variation in static point resistances and the Standard Penetration Test blow count with depth and continues to follow the previous mentioned relation of $q_o = 4N$ (ton/sq. ft.).

The investigations by Meyerhof in sand have also disclosed that for large displacement piles the unit skin resistance varies from 1.4 to 3 times that of the penetrometer due largely to the development of higher pressure against the side of the pile.

CHAPTER III

THEORETICAL BEARING CAPACITY

The total bearing capacity of a friction pile, represented in these tests by the penetrometer, is composed of two distinct parts. These are the point resistance and the skin resistance.

The point resistance of a pile is analogous to the bearing capacity of a simple spread footing; that is, it is derived from the vertical soil support offered to the end of the pile. When this vertical support yields, the soil particles move in radial planes away from the pile tip. If the soil is sufficiently dense, failure planes similar to Figure 1 are assumed to develop. On the other hand, if the soils are loose and compressible, failure planes similar to Figure 2 are assumed to develop. The soils dealt with in this study appear to fail as in Figure 2.

As outlined by K. Terzaghi (19) the unit soil support on a flat area is given by the following equation:

$$q_{ult} = cN_c + q'N_q + \frac{1}{2}\gamma bN_\gamma$$

The factors N_c, N_q, N_γ depend only on the angle of internal friction of the soil.

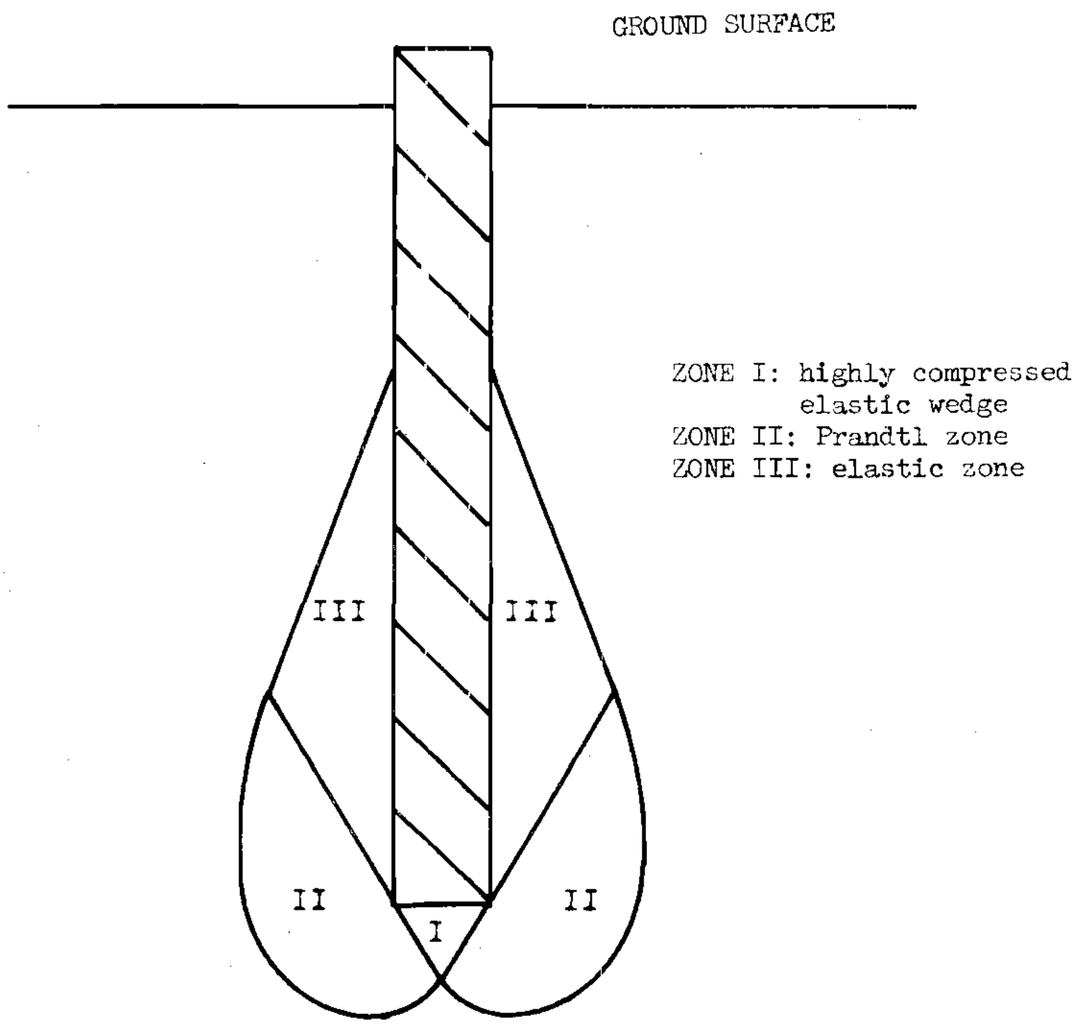


Figure 1. Assumed Bearing Capacity Failure Planes for Piles in Dense Soil

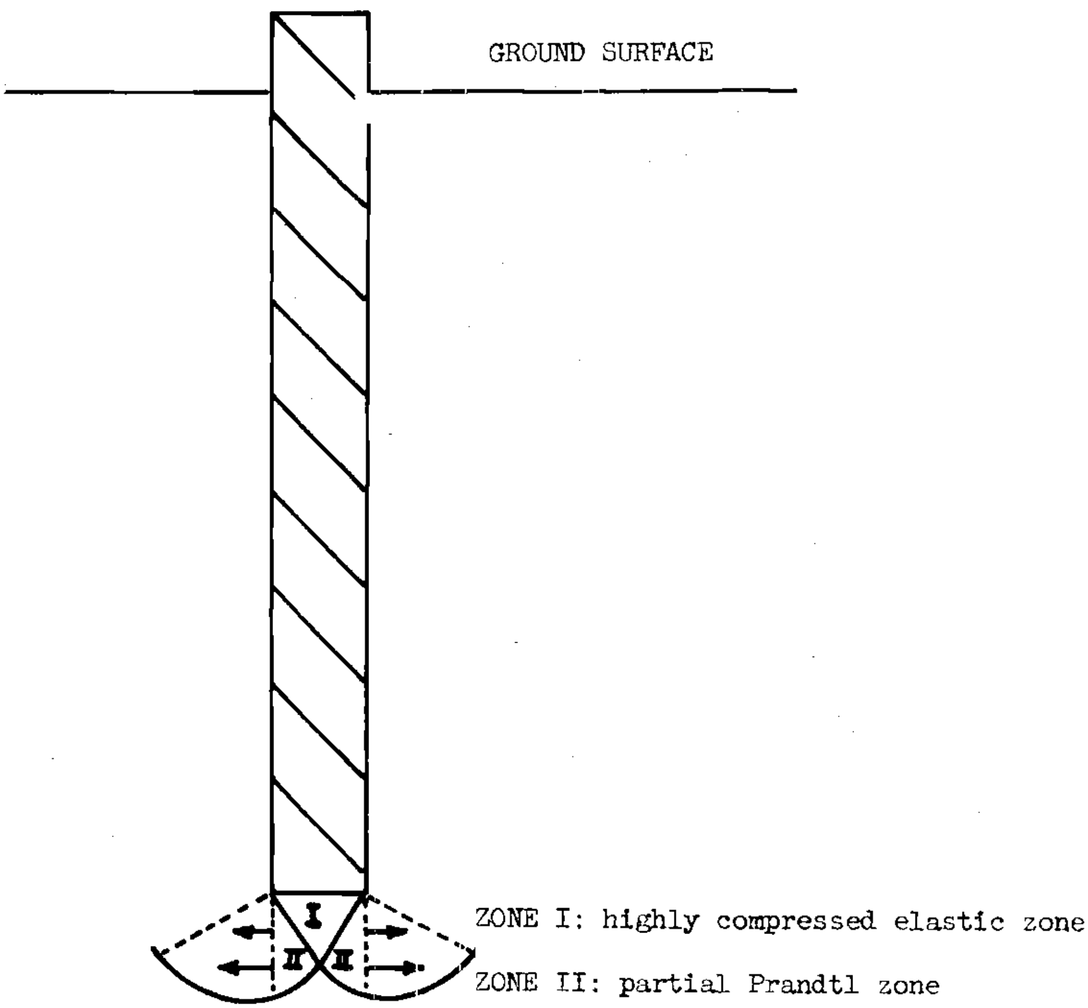


Figure 2. Assumed Bearing Capacity Failure Planes in Loose Soil (Local Shear Failure)

The values of these "N" factors have been determined both experimentally and theoretically by many investigators. In addition, shape factors, ξ_c, ξ_q, ξ_γ are used in the general bearing capacity equation to include the effect of changing from the theoretically infinitely long strip footing, used in the derivation of the bearing capacity equation, to a finite dimensional footing. The most important factors in the bearing capacity equation are the "N" factors.

Widely accepted values of N_c and N_q have been derived from work by Prandtl (1921) and Reisner (1924). Among different proposed values of N_γ, those computed by Caquot and Kerisel (1953) have been used for computing the theoretical bearing capacities in this thesis.

It has been proposed by Ivor Evans (6) that the bearing capacity of a loaded circular cone at the ground surface can be expressed as follows:

$$q = cM_c + \frac{1}{2}\gamma bM_\gamma$$

He has shown that the values of M_c and M do not differ greatly from the values of N_c and N_γ. However, for lower values of ϕ, M_c is lower than N_c and for higher values of ϕ they are about the same.

From this Evans concluded that the unit end bearing of a flat circular area is approximately the same as that for a circular conical surface. He does include the increase in unit bearing capacity due to increases in footing width.

The bearing capacity factors have been determined for both shallow and deep conditions. However, many investigators have questioned the validity of deep factors as it appears impossible for the assumed failure planes to develop. This study attempts to determine if "deep" bearing capacity factors are valid and what shape factors should be used with the general bearing capacity equation for the micaceous silts under investigation.

The skin resistance developed by the penetrometer, again analogous to a pile, is composed of two parts, as shown in Figure 3. The first part depends upon the actual adhesion of the soil to the pile, and the second part depends upon the friction developed between the soil and the pile due to lateral earth pressure.

The adhesion to the pile of the soil is directly related to the soil cohesion. This investigation shall try to determine what per cent of the cohesion should be used in bearing capacity analysis.

The true frictional resistance on the sides of a pile depends on the coefficient of lateral earth pressure, K_s, and the angle of friction, δ , between the soil and the pile. For analysis of data in this investigation the values of

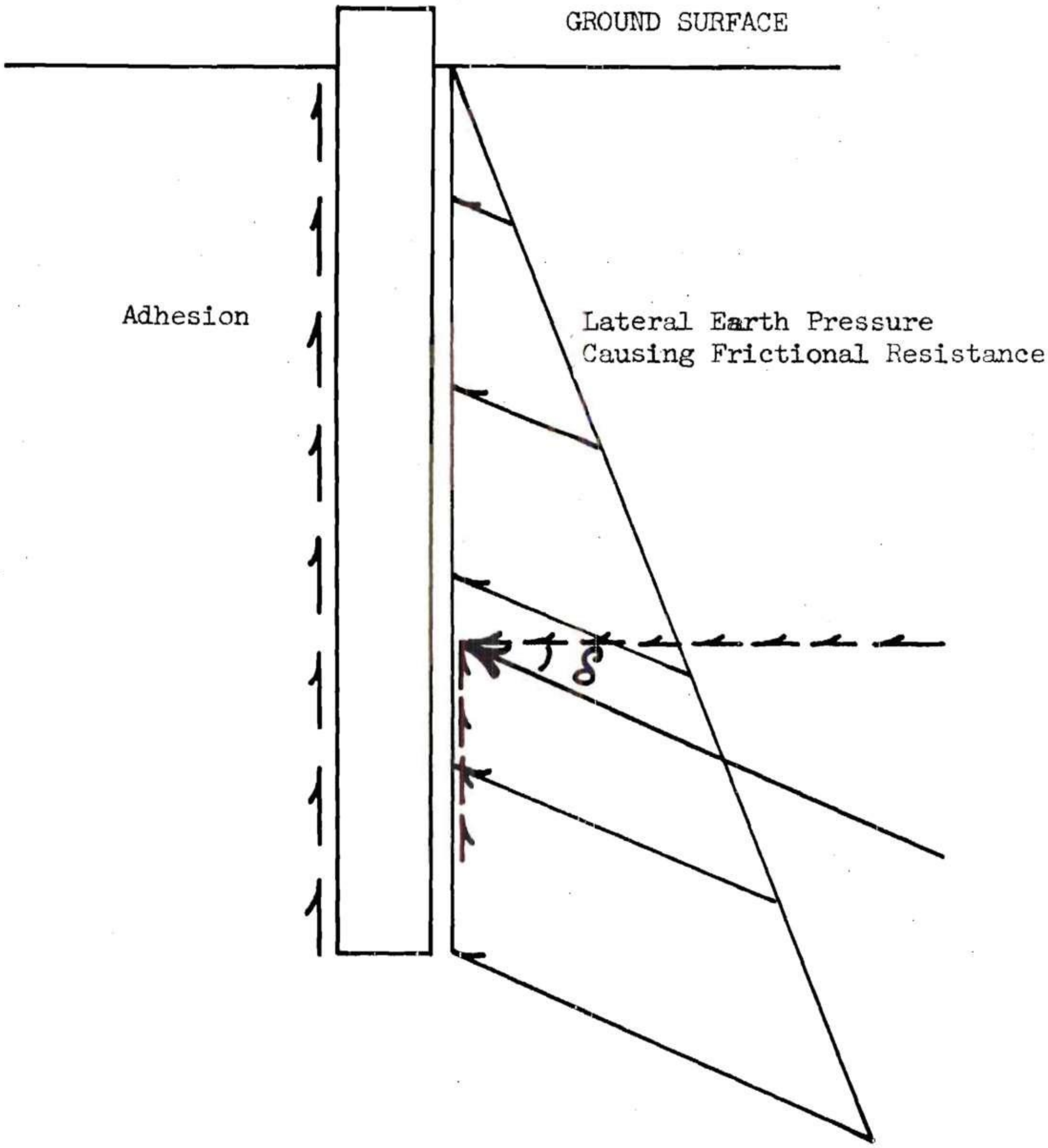


Figure 3. Adhesion and Friction Forces Acting on a Pile Shaft

equal to ϕ and to $2/3\ \phi$ are assumed. A coefficient of lateral earth pressure, K_s, equal to unity is used.

The next three sections of this study will describe the actual testing employed for the investigations. This portion shall include the design and construction of the static penetrometer, a description of the test sites, and a complete description of the field and laboratory testing procedures.

CHAPTER IV

DESIGN AND CONSTRUCTION OF THE PENETROMETER

The static penetrometer used in this study is fashioned after the type developed by the National Geotechnical Institute of Belgium. It is designed in such a way that point resistances and casing resistances may be measured separately. Thus, two complementary resistance diagrams are obtained from one sounding.

The point, or cone, of the penetrometer is formed from a 1 5/8 inch diameter cold rolled steel bar. The cone forms a central angle of 60°. The rear five inches of point assembly has been turned down 1/16 of an inch smaller in diameter to try to eliminate a majority of the skin resistance on the point. The sliding point assembly has a clearance of 1/64 of an inch and has proven sufficient to prevent the entrance of soil inside the point assembly which could hamper the operation.

The detailed dimensions of the point are shown in Figure 4.

The casing portion of the penetrometer is constructed from standard 1 5/8 inch diameter "A" drill rod. This drill rod is used in lengths of two and one half feet so as only short portions are unbraced above the ground when the load-

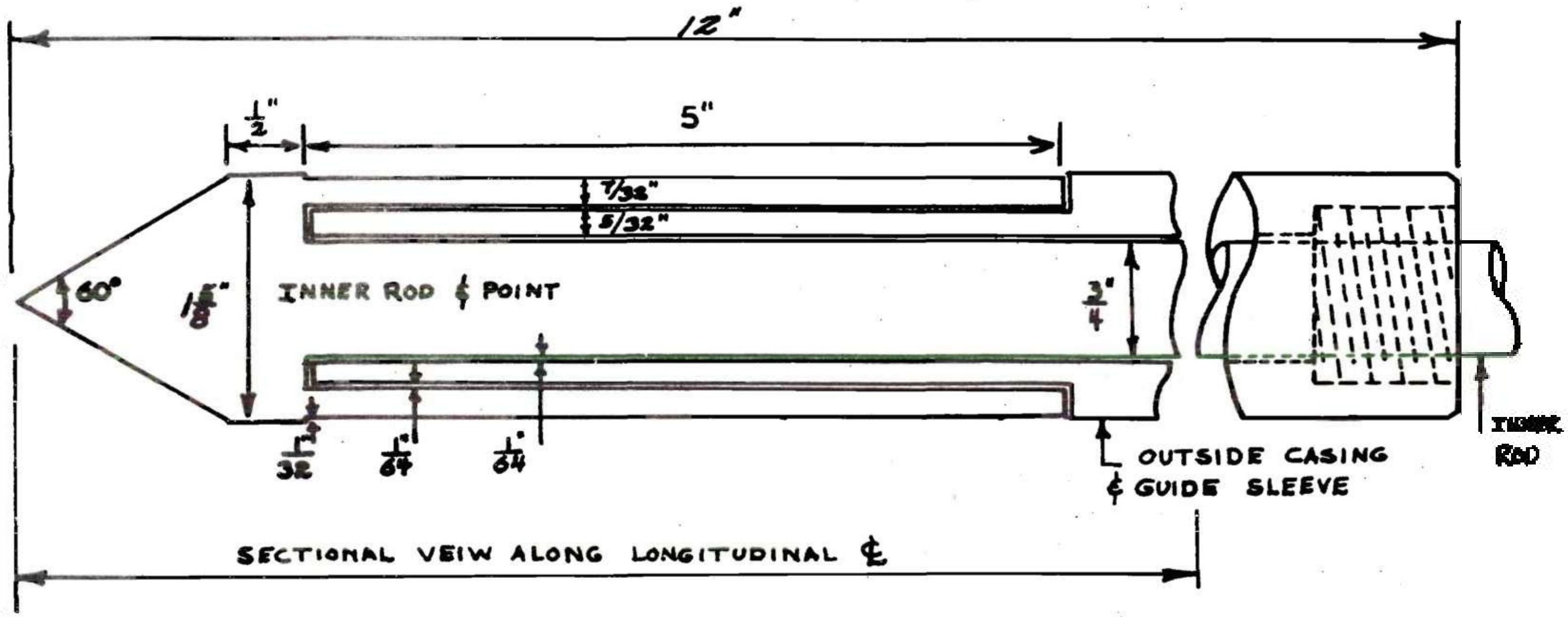


Figure 4. Details of Penetrometer Point

ing is in progress. The drill rod couplings were drilled out to an inside diameter of 25/32 inches. This was necessary to allow the inner rods to pass through. The inner rods, or the rods connected to the point, were formed with male and female screw threads designed for flush coupling. These are shown in Figure 5.

The attachments for pushing both the inner rods and the outside casing are shown in Figure 6. These were formed from the same stock as was the point assembly. Both the caps for the inner rod and the casing allow pushing without damage to the threads.

From these dimensions, the following constants have been found useful in calculation of results:

(1) Hydraulic pressure gage reading x 10.602 = pounds.

(2) Point resistance gage reading x 5.111 = psi.

(3) End area of cone equals 2.074 sq. in.

(4) Surface area of shaft equals 61.26 sq. in. per foot.

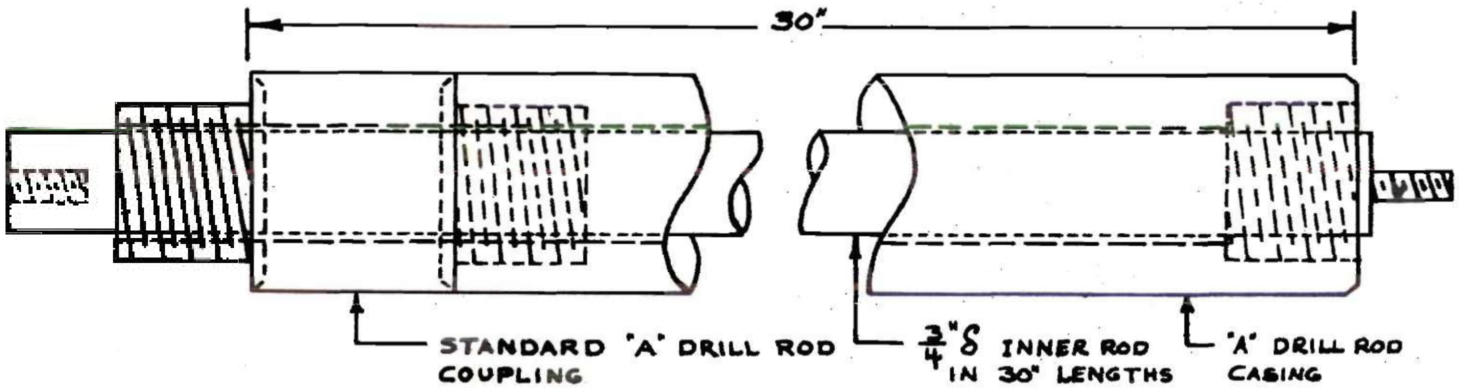


Figure 5. Details of Penetrometer Shaft

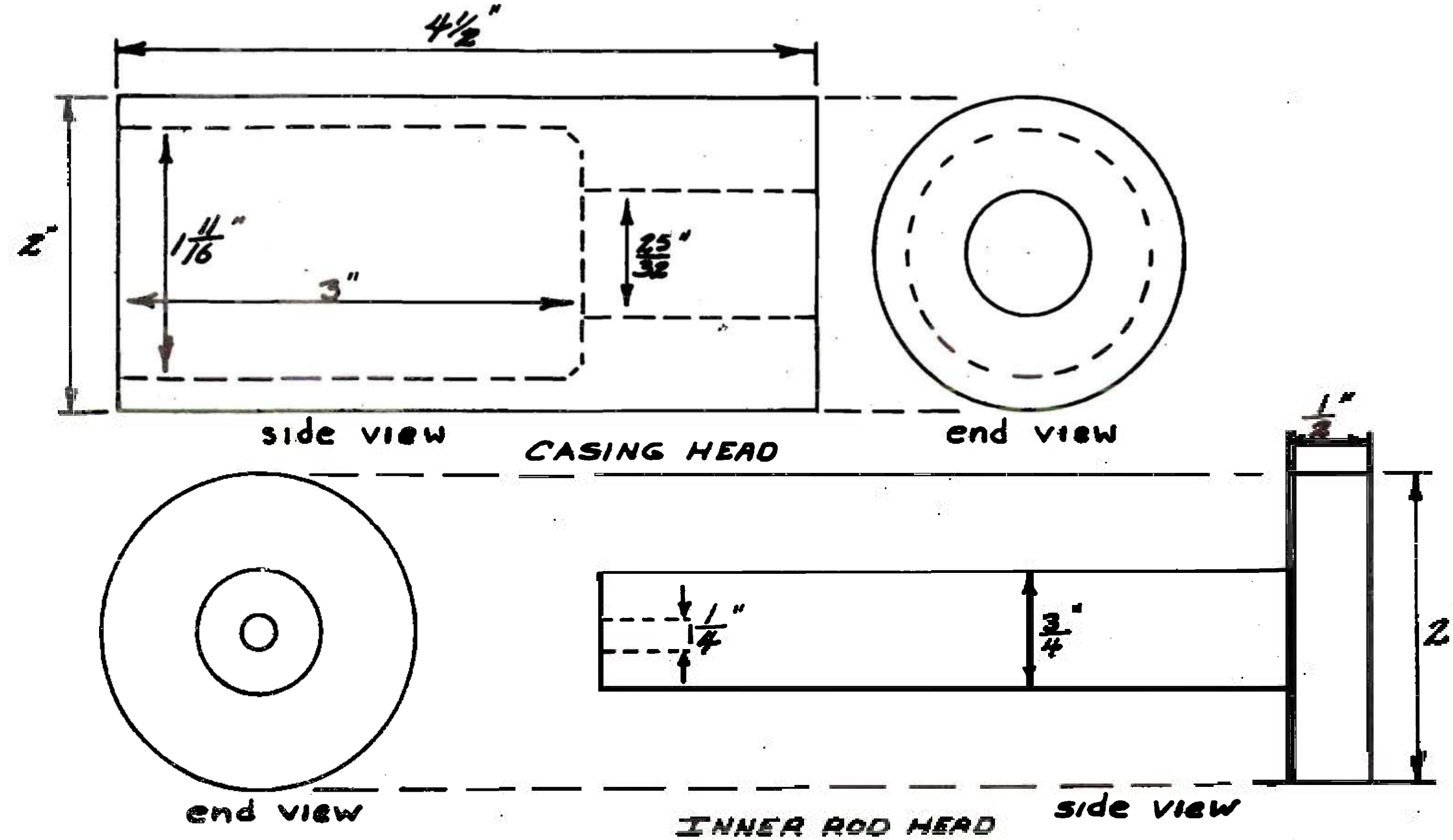


Figure 6. Details of Penetrometer Pushing Heads

CHAPTER V

DESCRIPTION OF TEST SITES

Site No. 1. This test site is the proposed location of a new pedestrian and automobile bridge on the University of Georgia campus. The bridge is to be founded in an area of old fill material. The penetrometer tests were performed at the center line of the bridge at bents Nos. 5 and 6 as shown on the preliminary layout by the Bridge Division of the Georgia State Highway Department.

Site No. 2. This test site is the location of the mainline bridges, crossing over State Route 164 in Banks County, of highway I-85. The bridges are to be founded in the natural ground at this site. The penetrometer tests were performed only for the North Bound Lane bridge at bents Nos. 2 (boring No. 70 on right edge of bridge) and 3 (boring No. 71, left side, and boring No. 72, on right side of bridge.)

Site No. 3. This test site is the location of the bridge for State Route 15 crossing over highway I-85 in Banks County. The area is in a small valley. The penetrometer tests were performed at bent No. 2 (sounding No. 2), bent No. 3 (sound No. 4), and bent No. 3 (sounding No. 6). All soundings were at the location of the left column for the bents.

Site No. 4. This test site is the location of the proposed new Cobb County Courthouse in Marietta. This site is approximately 150 yards northeast of the present Cobb County Courthouse. The penetrometer test was performed at the southwest corner (beneath the proposed corner column) of the new location.

Site No. 5. This test site is the proposed location of the bridge for the Atlantic Coast Line Railroad crossing highway I-85 near Fairburn in Fulton County. The area is generally level. The penetrometer tests were performed at points offset 10 feet south of outside right columns at bents Nos. 3, 4, and 5, as shown on the proposed layout plans prepared by the Bridge Division of the Georgia State Highway Department.

Site No. 6. This test site is location of the State Route 12 - Walnut Grove Road bridge over the Georgia Railroad line just east of Conyers in Rockdale County. The penetrometer tests were performed on the bridge center line at the two end bents.

NOTE: The subsurface conditions at all of the above sites are described on the boring logs adjacent to the penetrometer sounding logs.

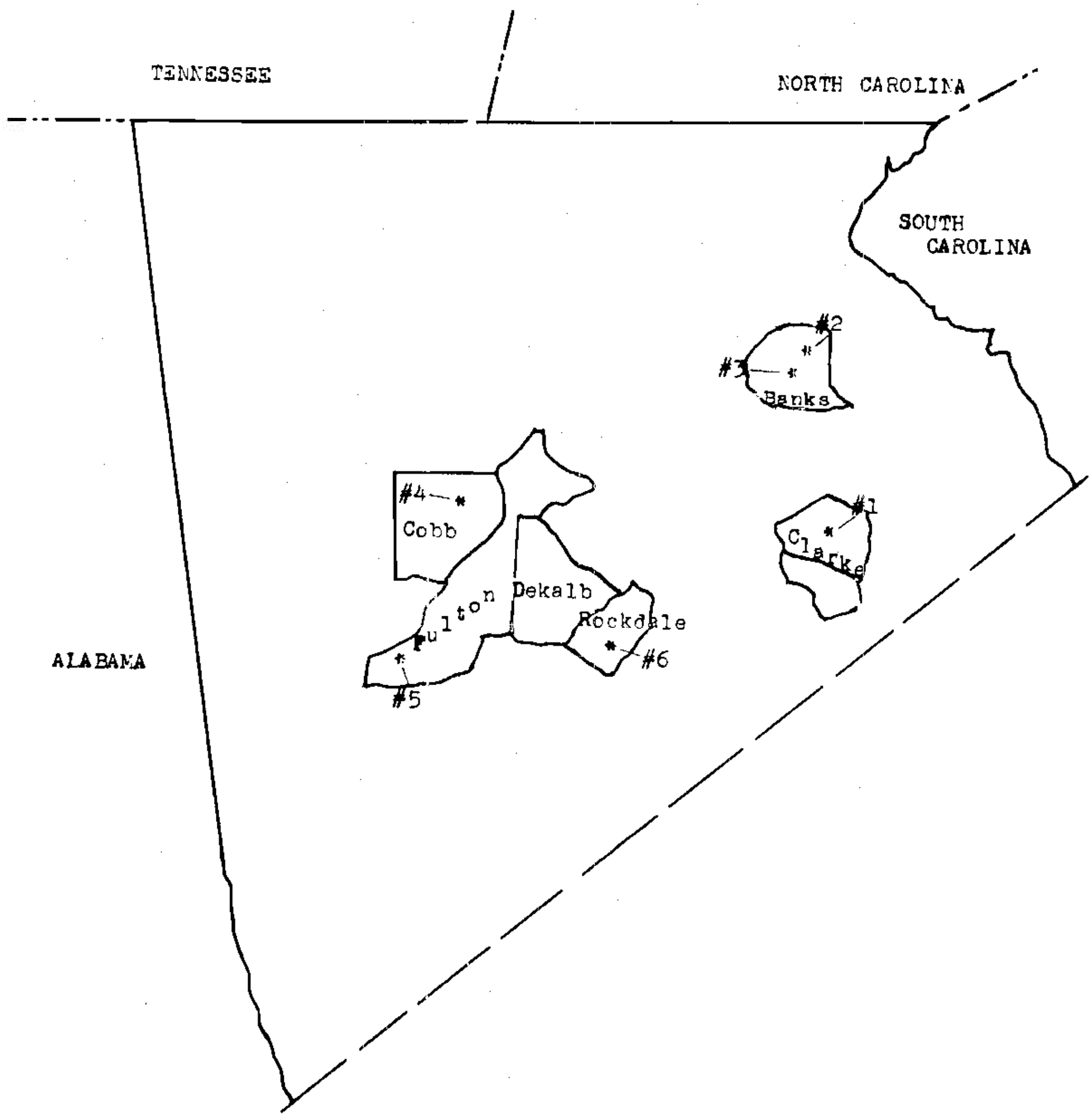


Figure 7. Map of North Georgia Showing Penetrometer Test Sites by County and Number

CHAPTER VI

TESTING PROCEDURE

The drill truck and hydraulic system used for these tests was a B-61 Mobile Drill belonging to the Georgia State Highway Department. This model drill was picked to be used for the static penetration testing because of its stronger hydraulic system and extra weight.

The modifications necessary for the drill before performing the penetration test consist of attaching a new, larger hydraulic pressure gage to the drill's hydraulic system and installing a manually operated coring chuck as this drill is used essentially with a universal joint auger chuck.

A two-and-one-half-foot auger with a flat plate welded to the lower end is placed in the chuck to provide a rigid pushing force closer to the ground.

When a sounding with the penetrometer is to be made, the point is first assembled with a five-foot section of outside casing and inner rod and pushed into the ground. This places the point below any surface disturbance and the testing is ready to begin.

At this position in the ground, the point is pushed three inches by means of the inner rod and the pressure recorded. Next the outside casing is pushed three inches and the pressure recorded. Then the total assembly is pushed nine inches and the pressure recorded. This procedure allows point resistance and skin resistance readings for every foot of depth penetrated.

The above procedure is followed until the desired depth is reached or to the ultimate capacity of the drill's hydraulic system (approximately five tons).

After the static penetrometer sounding has been made, the truck is moved approximately three feet away and an auger boring accompanied by Standard Penetration Tests and undisturbed sampling is performed. This is for correlation of soil properties and the Standard Penetration Test with the static penetrometer sounding.

The undisturbed samples (taken with a three-inch diameter thin-walled sampler) are taken to the laboratory for determination of the soil properties.

The in-place density is determined from the weight and dimensions of the sample as it is taken from the sampling tube. The moisture content and specific gravity are determined as outlined by ASTM.

The shear strength parameters, Ø and C, are determined in undrained or "quick" triaxial compression tests. In this test samples approximately 6 inches long and 2 27/32 inches in diameter are used. Confining pressures of one, three, and five Kips per square foot are used for the construction of Mohr's Envelope (see Appendix).

It should be noted that some of these envelopes showed a distinct curvature. In all cases the parameters Ø and C were obtained from the initial portion of the envelope.

The void ratio, "e," of the soil were computed for samples taken from beneath the ground water table by assuming complete saturation.

The ground water table was determined by measuring the height of water in the auger hole 24 hours after drilling.

CHAPTER VII

TEST RESULTS - END BEARING CAPACITY

The bearing capacity of the soils are measured in place by the resistance offered to the penetrometer. As proposed by Ivor Evans (6), it shall be assumed that the resistance offered to this cone-shaped "footing" is very nearly the same as would be offered to a flat footing.

These soundings show that the bearing capacity of the residual soils in the geographic area outlined decrease with depth until the footing elevation becomes near that of the lesser weathered rock. This decrease in bearing capacity may be attributed to:

(1) The upper soil strata are more weathered and contain a higher percentage of clay, thus giving added strength to the soil.

(2) Dessication at the surface has tended to densify the upper soils.

(3) The lower strata of soils generally have a higher void ratio, thus indicating a lower angle of friction.

(4) The buoyant effect of the ground water table as well as possibly some seepage softening of these soils.

This investigation has also shown that the use of deep bearing capacity factors similar to those proposed by Meyerhof are not valid for use in these soils.

Table 1 gives the values of measured unit bearing capacity from the point resistance and the theoretical bearing capacity as computed, using Prandtl-Reissner factors and shape factors equal to one. The third term, $\frac{1}{2}\gamma b N_{\gamma}$, of the bearing capacity equation is also omitted.

The values of Table 1 are also presented in graphical form in Figure 8. The theoretical and measured resistances compare very well, thus leading to the conclusion that Prandtl-Reissner bearing capacity factors and shape factors equal to one should be used for pile analysis in these soils.

In all cases where bearing capacity was computed, the ratio of depth to footing width was greater than 10, thus fulfilling the requirements for the use of deep bearing capacity factors. In all cases shallow factors were used, giving the results of Table 1. These results were sometimes slightly higher and sometimes slightly lower than the measured resistances. This makes it difficult to determine if the use of shape factors equal to one is valid. The results are definite enough to establish that the use of shape factors equal to one for piles will give reasonably close results.

Table 1. Theoretical and Measured Point Resistances

Site and Boring No.	Depth (Ft.)	Measured Point Resistance psi	Theoretical Point Resistance psi
SR-15 #2	8	306	349
SR-15 #2	14	128	126.7
SR-15 #2	19	153	140.8
SR-15 #2	23	153	144.0
SR-15 #4	9	255	261.0
SR-15 #4	13	460	416
SR-15 #6	10	357	149
SR-15 #6	13	322	94
SR-164 #71	8	153	154.7
SR-164 #71	18	102	127
SR-164 #72	9	178	149
SR-164 #72	13	127	109
SR-164 #72	18	76.6	93
SR-164 #70	22	152	136
SR-164 #70	18	25.5	26.3
ACL RR #7	10	76.6	77.7
ACL RR #7	15	178.0	95.2

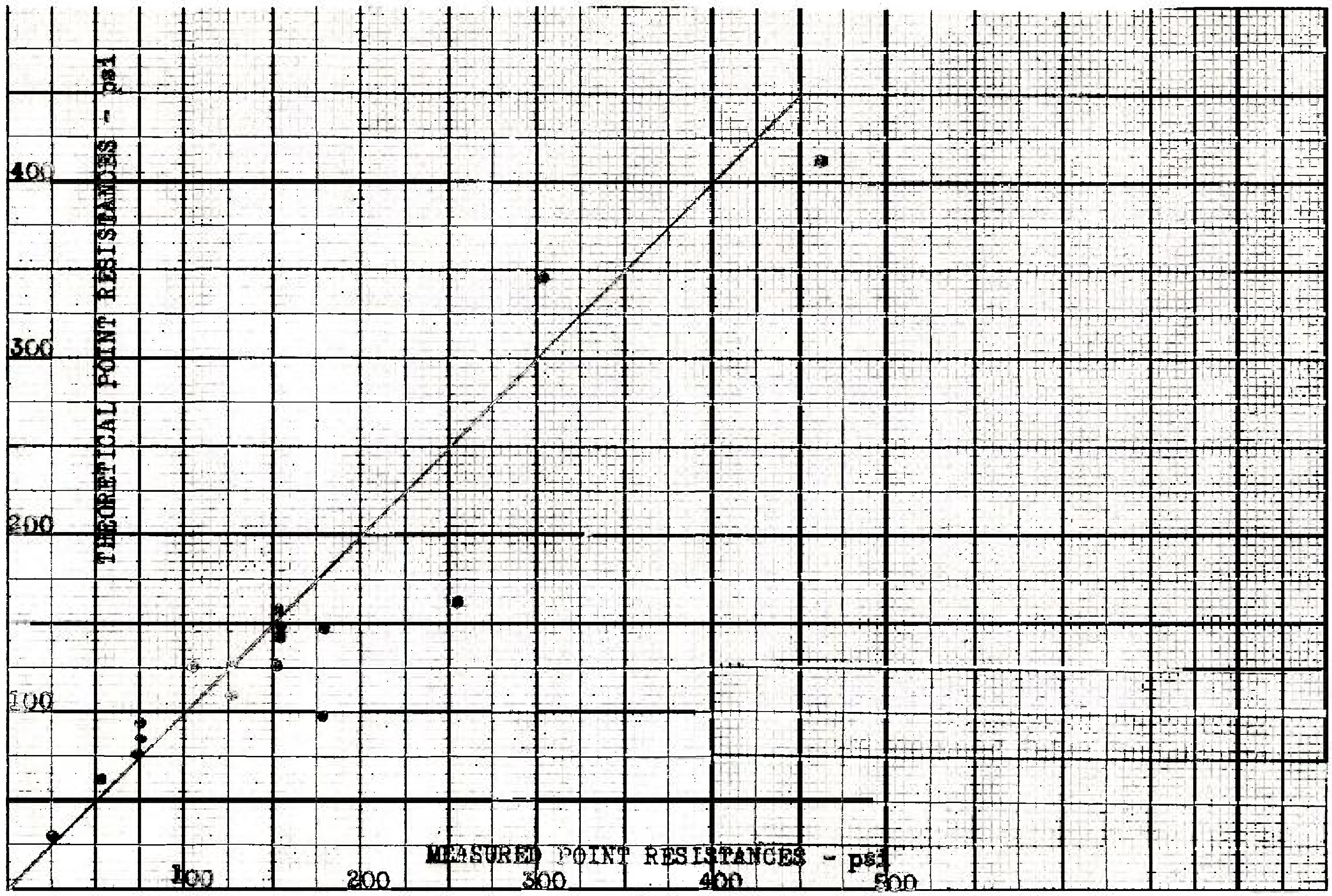


Figure 8. Relation of Theoretical and Measured Point Resistances

CHAPTER VIII

TEST RESULTS - RELATION OF ADHESION TO SOIL COHESION

Another of the objectives of this study was to try to determine what percentage of the soil cohesion is active in supporting a friction pile by adhering to the pile skin.

It is generally understood that pile driving causes remolding and subsequent loss of strength in some soils. Therefore the most critical time in the supporting requirements of the soil would be immediately after driving the pile. The values of the adhesion used in this study were established and measured while the penetrometer was in motion.

As previously stated, the undisturbed samples have been tested for undrained shear strength. The strength parameter "c" is the quantity used when the soil cohesion is referenced in this discussion.

The measured values of increases in casing resistance over a specified depth have been reduced to values of average casing resistance over various depths in pounds per square foot of shaft area. From the soil properties, the lateral earth pressure offering resistance to penetration has been computed for the following assumptions:

(1) $K_s = 1$; angle of wall friction, δ, $= 2/3\ \phi$.

(2) $K_s = 1$; angle of wall friction, δ , = $\emptyset$.

The actual adhesion of the soil to the penetrometer has been computed from the following equation by subtracting the lateral earth pressure resistance from the total casing resistance.

Casing resistance (psf) = adhesion + $K_s \gamma Z \tan \delta$

or adhesion (psf) = casing resistance (psf) - $K_s \gamma Z \tan \delta$.

The relation of the adhesion to the cohesion is expressed by m equal to adhesion/cohesion. Table 2 also gives the values of m computed assuming δ equal to 2/3 $\emptyset$ and δ equal to $\emptyset$. The values computed for δ equal to $\emptyset$ appear to be more in reason, as there are fewer values greater than one. An average value for m with δ equal to $\emptyset$ is 0.66. The values of m computed for δ equal to 2/3 $\emptyset$ average to be 0.60 if all values greater than 1.2 are omitted.

This analysis shows that an assumption of a set of values of K = 1, and δ = $\emptyset$ can explain the observed relation of M = 0.66.

Table 2. Comparison of Theoretical and Actual Adhesion

Location and Boring #		Depth	Resistance (pounds)	Distance (feet)	Ø	C KSF	γ Z tan Ø	Adhesion δ =Ø	M δ =Ø	M δ =2/3 Ø
SR 15	#2	8-9	106	1	24	2.3	181	68	.03	.05
	#2	8-14	1378	5	19	1.15	181.5	465.5	.40	.46
	#2	14-17	1272	3	24	.7	305	692.6	.99	1.1
	#2	17-24	2969	7	22	.75	354	643.6	.86	1.01
	#4	6-9	954	3	27	1.15	200	548.2	.48	.54
	#4	9-13	2968	4	25	2.3	278	1467.8	.64	.68
	#6	8-10	1378	2	10	2.5	88	1533.1	.62	.62
	#6	10-13	5407	3	13	1.2	145	4095	3.4	3.5
SR 164										
	#70	15-20	2492	5	14	2.35	308	863	.37	.41
	#70	20-26	1590	6	21	.45	454	169	.38	.72
	#71	6-10	1962	4	15	1.7	109.2	1044.9	.62	.64
	#71	10-14	953	4	21	.35	241	319	.91	1.14
	#71	14-19	584	5	18	.35	279	-5.0	-	.25
	#71	19-21	689	2	14	.25	259	551.6	.68	2.56
	#72	7-10	1961	3	22	.9	174.5	1361.7	1.51	1.58
	#72	15-20	319	5	17	.75	305	-157.8	-	-
Univ. of Ga.	#6	5-10	371	5	10	.9	70.2	103.8	.12	.14
	#6	10-15	689	5	5	.6	56.4	267.9	.45	.48
	#6	15-20	1908	5	31	.3	518	244	.81	1.4
	#6	20-25	1803	5	20	1.2	427	420.1	.26	.45
ACL RR										
	#7	5-8	424	3	22	.4	134	198.4	.50	.61
	#7	8-13	3340	5	14	1.0	136.1	1434.9	1.44	1.48
	#7	13-18	3975	5	19	.6	273	1597.5	2.5	2.72

CHAPTER IX

TEST RESULTS - RELATIONSHIP OF POINT RESISTANCE TO STANDARD PENETRATION TEST BLOW COUNT

As outlined in the testing procedure, Standard Penetration Testing has been performed in drill holes adjacent to the penetrometer soundings.

The Standard Penetration Test is a dynamic shear test which is defined as the number of blows of a 140-pound hammer falling 30 inches required to drive a 1-1/2-inch-diameter sampler for 12 inches.

Figure 9 shows the graphical relation of the measured point resistances and the Standard Penetration Test blow count "N" in adjacent holes at corresponding depths. This plot has the expected scattering of points, due to many reasons, the major one of which is the inhomogeneity of the in-place soils.

The trend of these points can be represented by the solid line shown on the graph and expressed by the following relation:

$$q_o \text{ psi} = 33.3 \text{ N}$$

Possibly an exponential equation could be derived that would more closely approximate the relationship, especially in

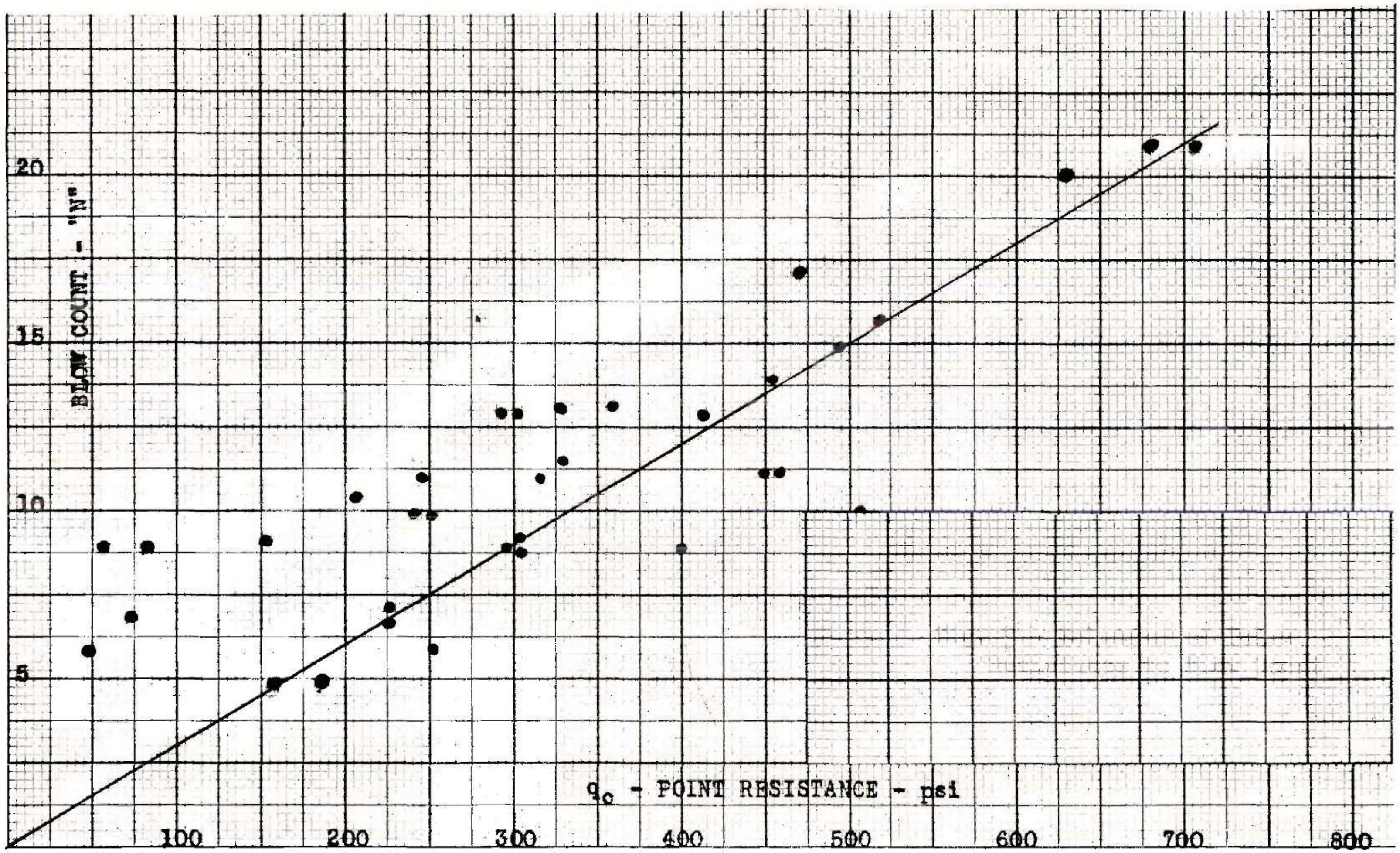


Figure 9. Relation of Point Resistance to Blow Count

range of N less than 5, but it is felt that the variation in test results warrants only the use of a simple equation.

The equation of end bearing capacity or point resistance of a pile may be expanded to the units of tons per square foot as in the following equation:

$$q_{tsf} = 2.4\ N$$

In this form the equation would be useful in calculating the end bearing capacity of a pile. It should be noted that the coefficient, 2.4, is between 4, proposed for sand, and 2, proposed for clay.

CHAPTER X

TEST RESULTS - RELATIONSHIP OF SKIN RESISTANCE TO STANDARD PENETRATION TEST BLOW COUNT

The correlation of the penetrometer casing resistance and the Standard Penetration Test blow count has been studied in two ways. First, the values of the increase in casing resistance per foot of depth penetrated is compared to the blow count, and secondly, the total casing resistance is compared to the average blow count throughout that depth.

The first relation, increase in casing resistance per foot and the blow count, is presented in Figure 10. Here again there is a considerable scattering of points. An average relation for these values may be expressed by:

$$\text{Increase in Resistance/Ft.} = 90N(\text{pounds})$$

This equation represents the solid line on the graph.

The second relation, total casing resistance and the blow count, is presented in Figure 11. Here the scattering of points is not as great as in the previous relation. The solid line shown on the graph as an average relation may be represented by:

$$\frac{\text{Total Casing Resistance}}{\text{Depth}} = Q_s/Z = 35N(\text{pounds/ft.}).$$

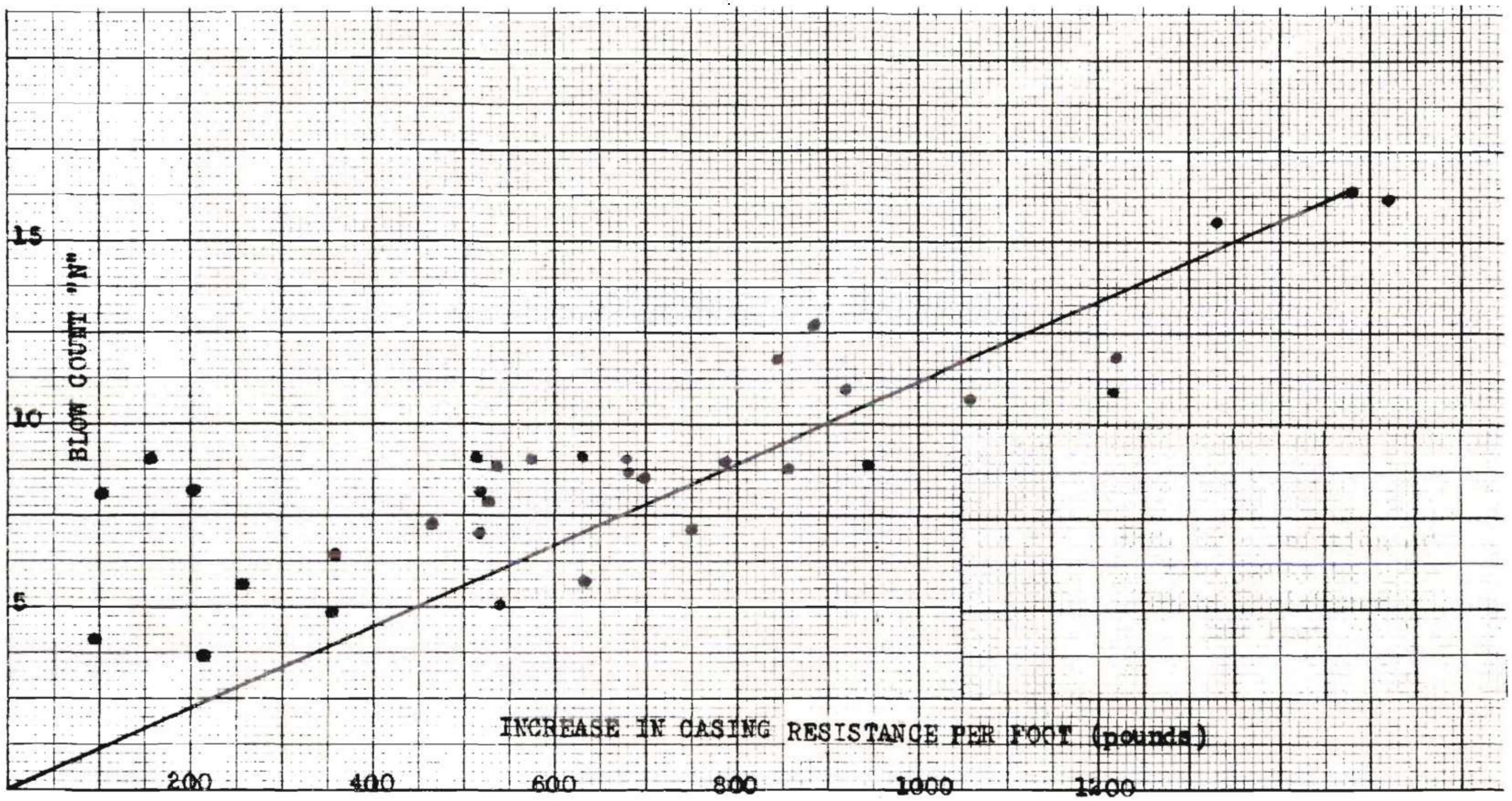


Figure 10. Relation of Blow Count to Casing Resistance Per Foot

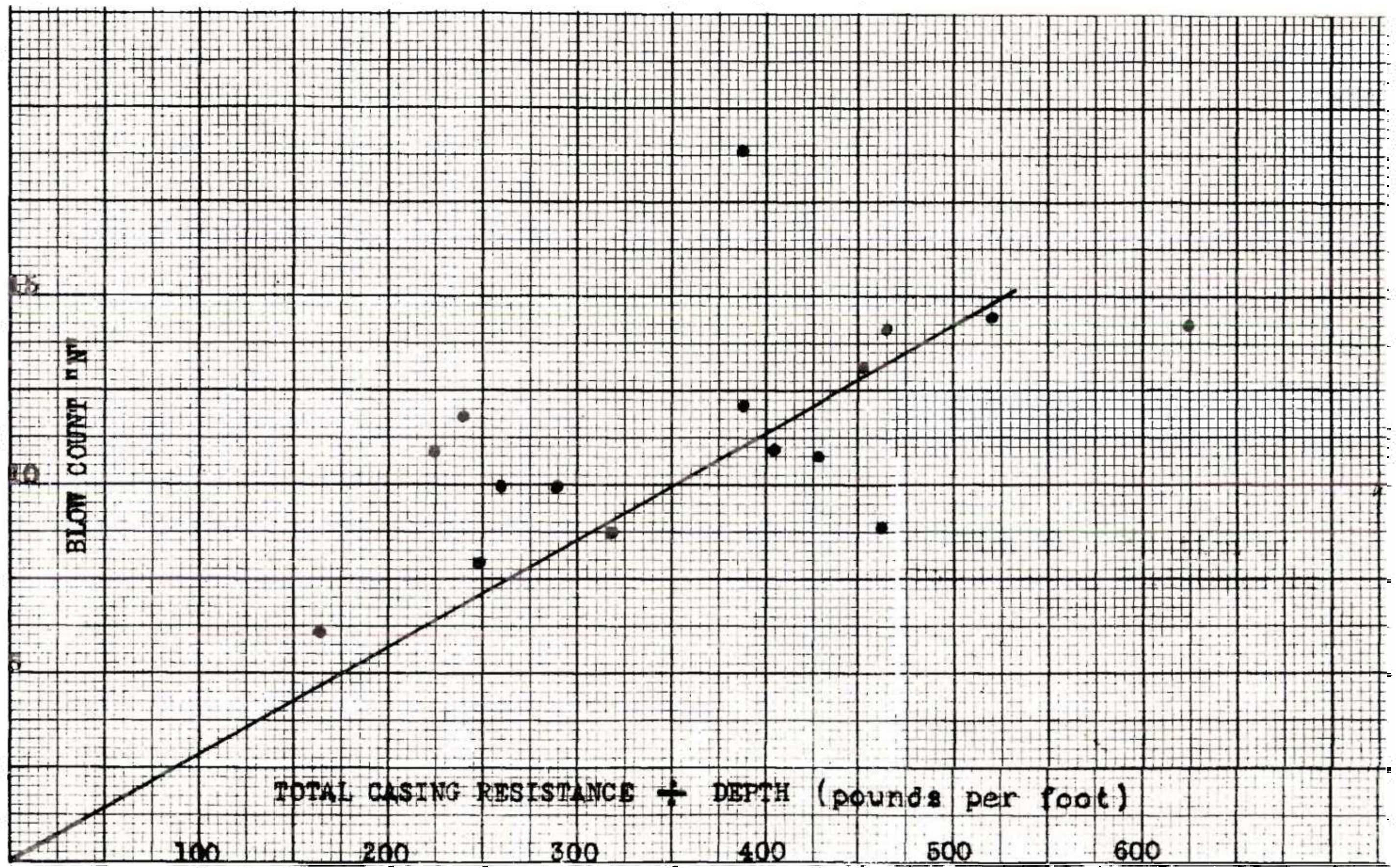


Figure 11. Relation of Total Casing Resistance to Blow Count

CHAPTER XI

PILE CAPACITY FROM PENETRATION TESTS

Many investigators have proposed relationships for pile bearing capacity based on static penetration tests using equipment similar to the subject of this thesis and also based on the Standard Penetration Test blow count.

One of the most common of these comparisons is relating the ultimate load on a pile with the ultimate load on a static penetrometer by means of the ratio of end areas and skin surface areas. This would be as follows:

$$Q_{ult.} \text{ of pile} = q_o A_p + Q_s \frac{A_s}{A_c}$$

where:

q_o = unit resistance to penetrometer point (psi)

Q_s = total penetrometer casing resistance within range of pile foundation (lbs.)

A_p = area of pile tip (sq. in.)

A_s = surface area of pile shaft (sq. in./ft. of length)

A_c = surface area of penetrometer shaft (sq. in./ft. of length)

For this relation to be true, the assumption must be made that the dimensions of the shear planes at failure are proportional to the pile or penetrometer diameters. However, field load tests by Meyerhof have shown that the observed skin resistance on driven piles in sand varies from one and one-fourth to three times observed skin resistance on the penetrometer. This suggests an average increase in unit skin resistance from a small diameter penetrometer to a large diameter pile of two.

Meyerhof has further concluded that the unit skin resistance for small displacement piles in sand, such as steel "H" sections, is the same as the unit skin resistance on the penetrometer.

From the previous references and assumptions, the following equation is offered for the bearing capacity of a pile in micaceous silts based on static penetration tests with a penetrometer designed as in Chapter IV.

$$Q_{ult.} = q_o A_p + (K f_s \, A_s \, Z) \quad \text{(Q in pounds)}$$

The meaning of the symbols are the same as before with the addition of:

f_s = unit casing resistance (psi)

K = coefficient for increase in lateral pressure due to larger displacement piles (K varies from 1 to 3).

It is hoped that load testing of several large diameter piles may be accomplished in the near future to determine the value of K. It is felt that the soils in question are compressible enough that the value of K is very near unity.

Another approach to a pile capacity expression has been by expressing the capacity of the penetrometer in terms of the blow count by the relations given in Figures 9 and 11. The capacity of the penetrometer may then be expanded as before to the capacity of a larger diameter pile.

The ultimate load carrying capacity of the penetrometer may be expressed as:

$$Q_{ult.} = 33.3 \; N \; A_c + 35 \; N \; Z \quad \text{(Q in pounds)}$$

This equation then expanded to a larger diameter pile based on proportional areas would be:

$$Q_{ult.} \text{ of pile} = 33.3 \; N \; A_p + 35 \; N \; Z \; A_s/A_c \quad \text{(Q in pounds)}$$

According to Meyerhof's findings, the latter equation would be valid for small displacement piles. However, until load tests of full size piles are accomplished, the equation should be used in the following form:

$$Q_{ult.} \text{ of pile} = 33.3 \; N \; A_p + K \; 35 \; N \; Z \; A_s/A_c \quad \text{(Q in pounds)}$$

where K is equal to one (1).

The term $\frac{A_s}{A_c}$ relates the surface areas of the proposed pile and the penetrometer.

An example application of the equation would be to compute the length of a 12-inch diameter pile necessary to develop 120 tons' capacity (40 tons with a safety factor of 3). The soil at the site has an average Standard Penetration Test blow count of 15.

Solving for Z or the depth from the equation, it is seen that:

$$Z = \frac{Q - 33.3\ N\ A_p}{K\ 35\ N} \cdot \frac{A_c}{A_s}$$

or Z equals to 47 feet.

CHAPTER XII

CONCLUSIONS AND PRACTICAL DATA

The design and use of the penetrometer in this thesis has been primarily for the determination of the practicality of static penetrometers in the Southern Piedmont region. The following major observations have been made:

1. A static penetrometer may be used with apparent success in this region, but a downward force of approximately 10 tons will be required for complete use.

2. The design of the penetrometer (both point and the rods) is sufficiently strong to facilitate consistent use. It is suggested that driving the penetrometer never be done. In addition to the above observations, valuable data has been gathered as to the bearing capacity of the micaceous silts.

These may be summarized as:

1. The use of Prandtl-Reissner (shallow) bearing capacity factors with a shape factor equal to one give very good comparisons with measured capacities, regardless of depth of the relation of footing width to depth.

2. A value of the ratio of adhesion to cohesion $\left[\frac{\text{adhesion}}{\text{cohesion}}\right]$ equal to .66 could be proposed for these soils, assuming that $K = 1$ and $\delta = \phi$ (in pile analysis).

3. There is a general relation of the penetrometer point resistance and the Standard Penetration Test blow count of

$$q_c = 33.3 \text{ N} \quad \text{(psi)}$$

4. There is a general relation of the Standard Penetration Test blow count to the casing resistance of the penetrometer of:

$$\frac{\text{Total Casing Resistance}}{\text{Depth}} = 35 \text{ N}$$

Increase in casing resistance per foot = 80 N

APPENDIX

PENETRATION DIAGRAMS

The following thirteen pages are the penetration diagrams for all soundings included in this thesis. Included on the diagrams are:

(1) a subsoil profile

(2) graphical representation of point and casing resistances

(3) graphical representation of blow count "N"

(4) shear strength parameters for soils encountered.

The point resistances are plotted in pounds per square inch and the casing resistances are plotted in pounds.

PENETROMETER LOG

Project No. & County : BA (5) SP 1645 Clarke

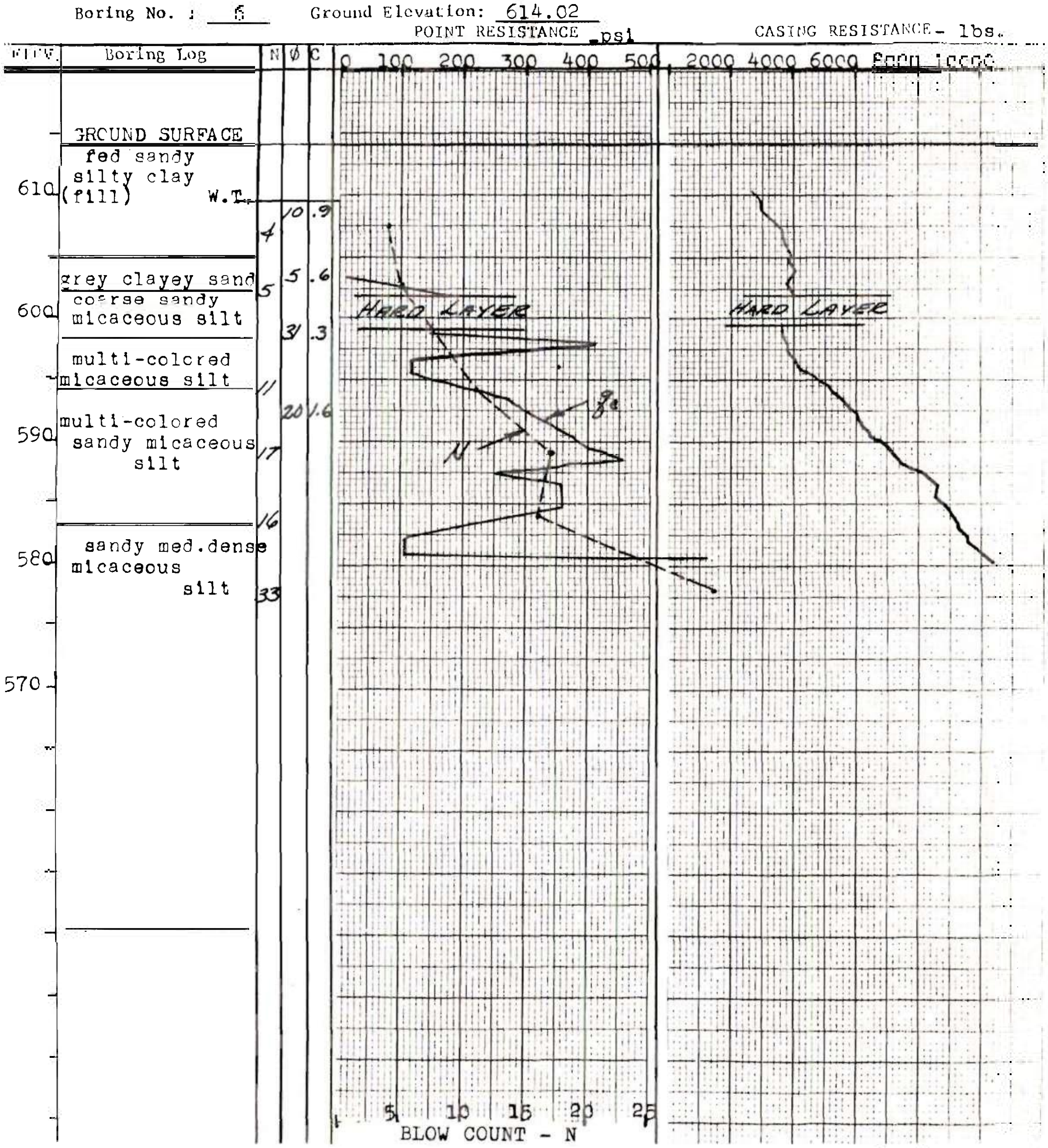
Party Chief : Crowther

Date : 2/19/62

Bridge Site & Station No. : Site #1 Univ. of Georgia campus

Boring No. : 5

Ground Elevation: 614.02



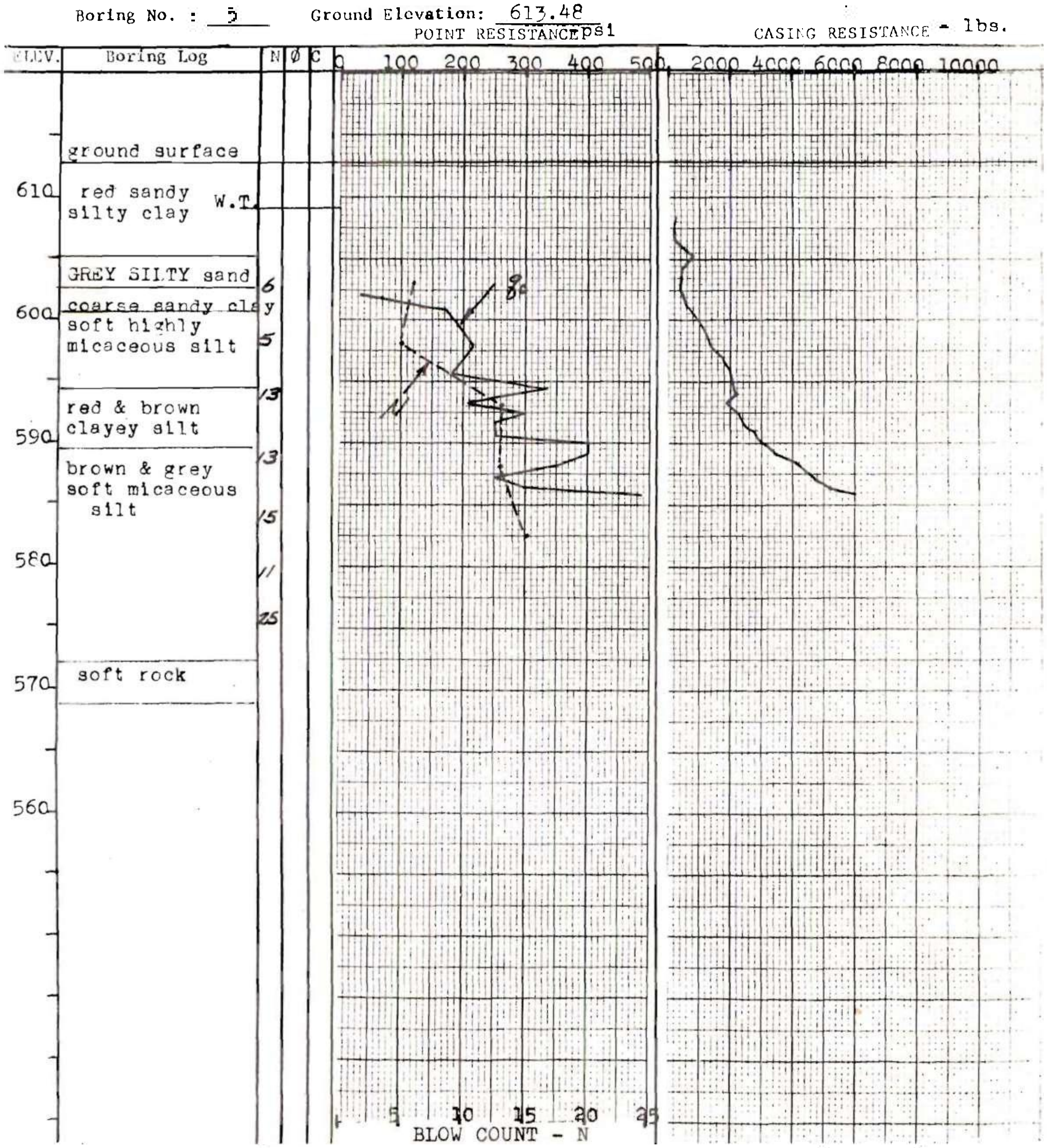
PENETROMETER LOG

Project No. & County : BA(5)SP 1645 Clarke

Party Chief : Crowther
Date : 2/19/62

Bridge Site & Station No. : Site # 1 Univ. of Georgia campus

Boring No. : 5 Ground Elevation: 613.48



PENETROMETER LOG

Project No. & County : I-85-2(5) Banks

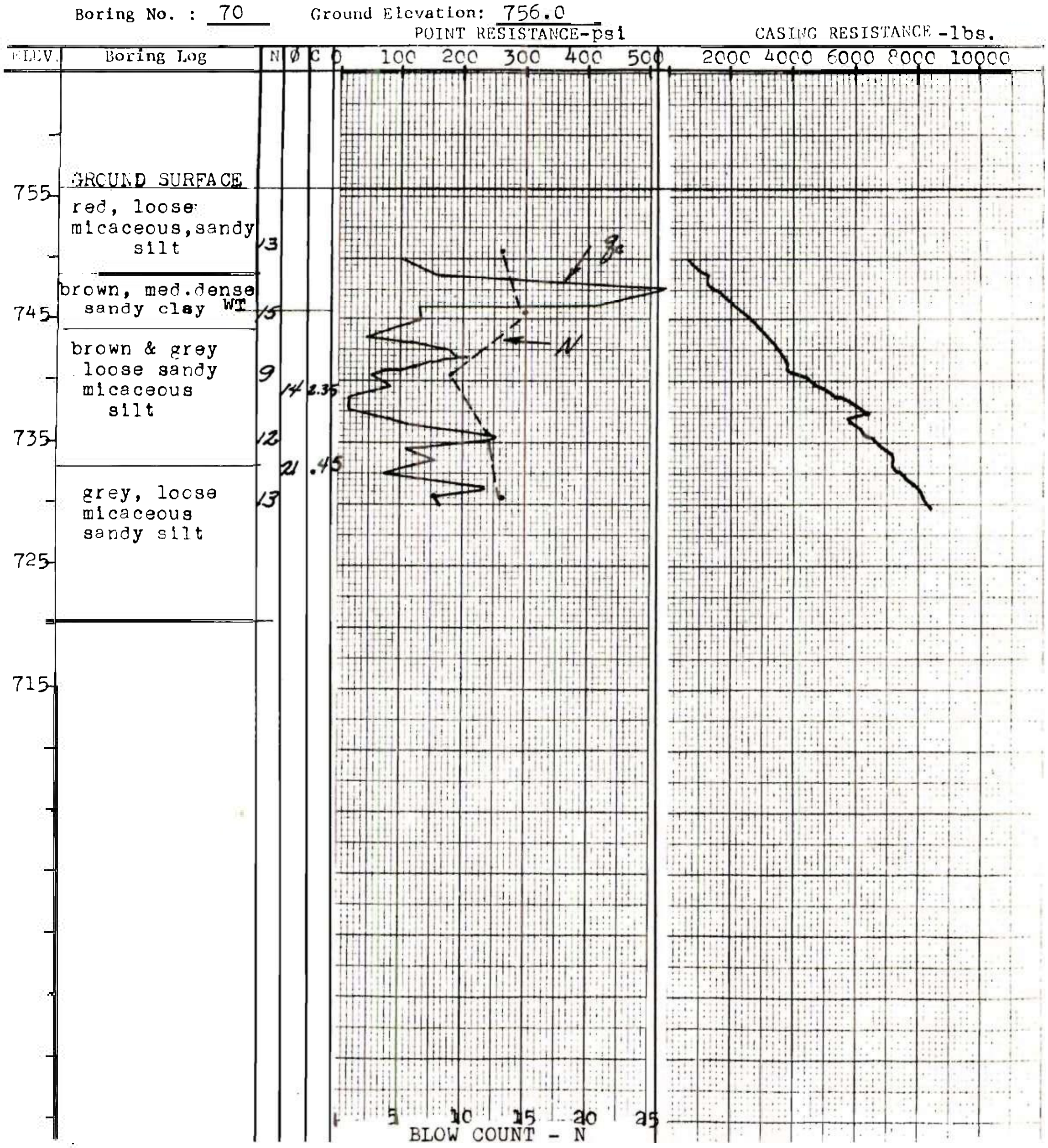
Party Chief : Crowther

Date : 8/30/62

Bridge Site & Station No. : Site #2 SR 164

Boring No. : 70

Ground Elevation: 756.0



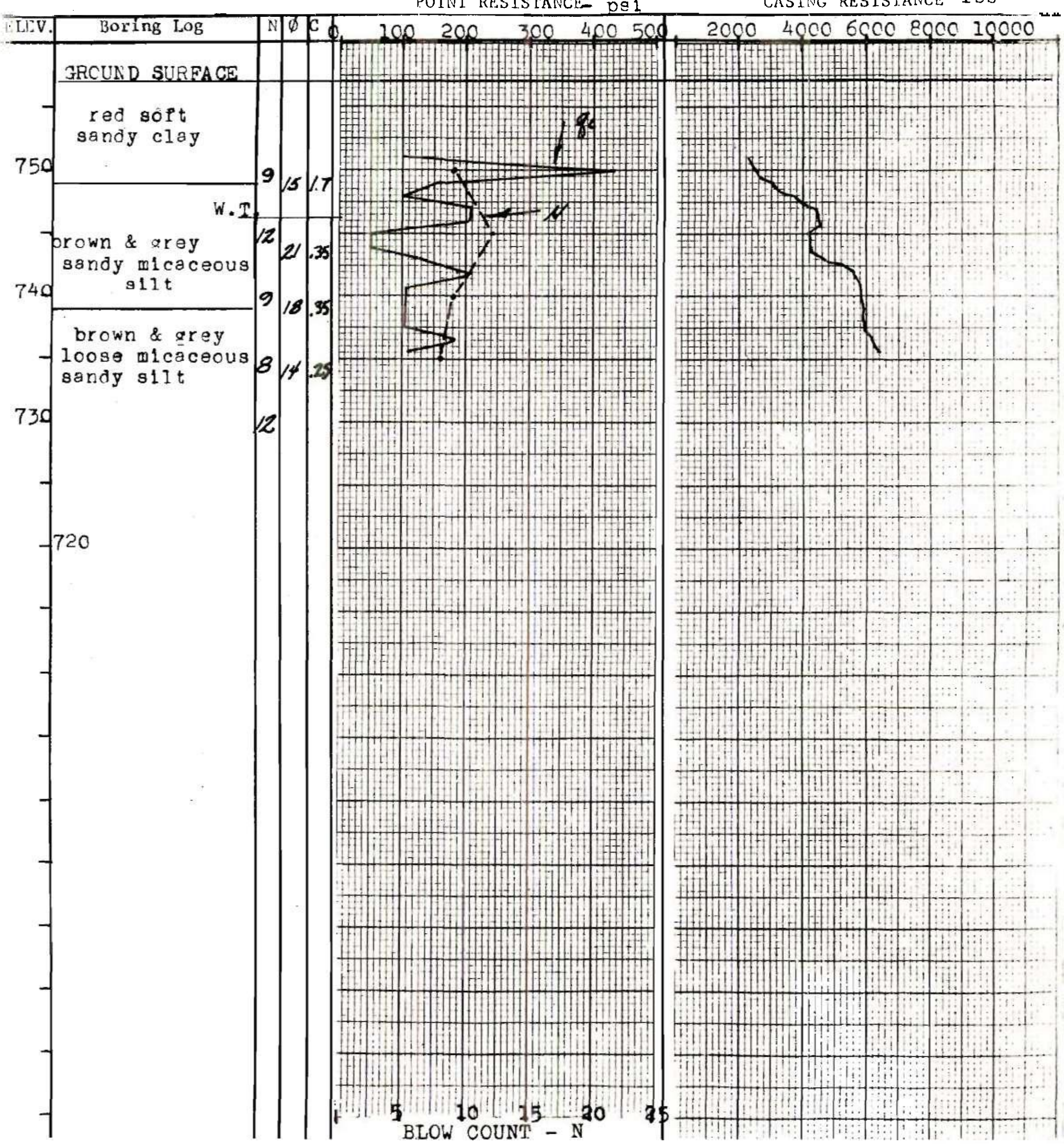
PENETROMETER LOG

Project No. & County : I-85-2(5) Banks

Party Chief : Crowther
Date : 8/27/62

Bridge Site & Station No. : Site #2 SR 164

Boring No. : 71   Ground Elevation: 757.1



PENETROMETER LOG

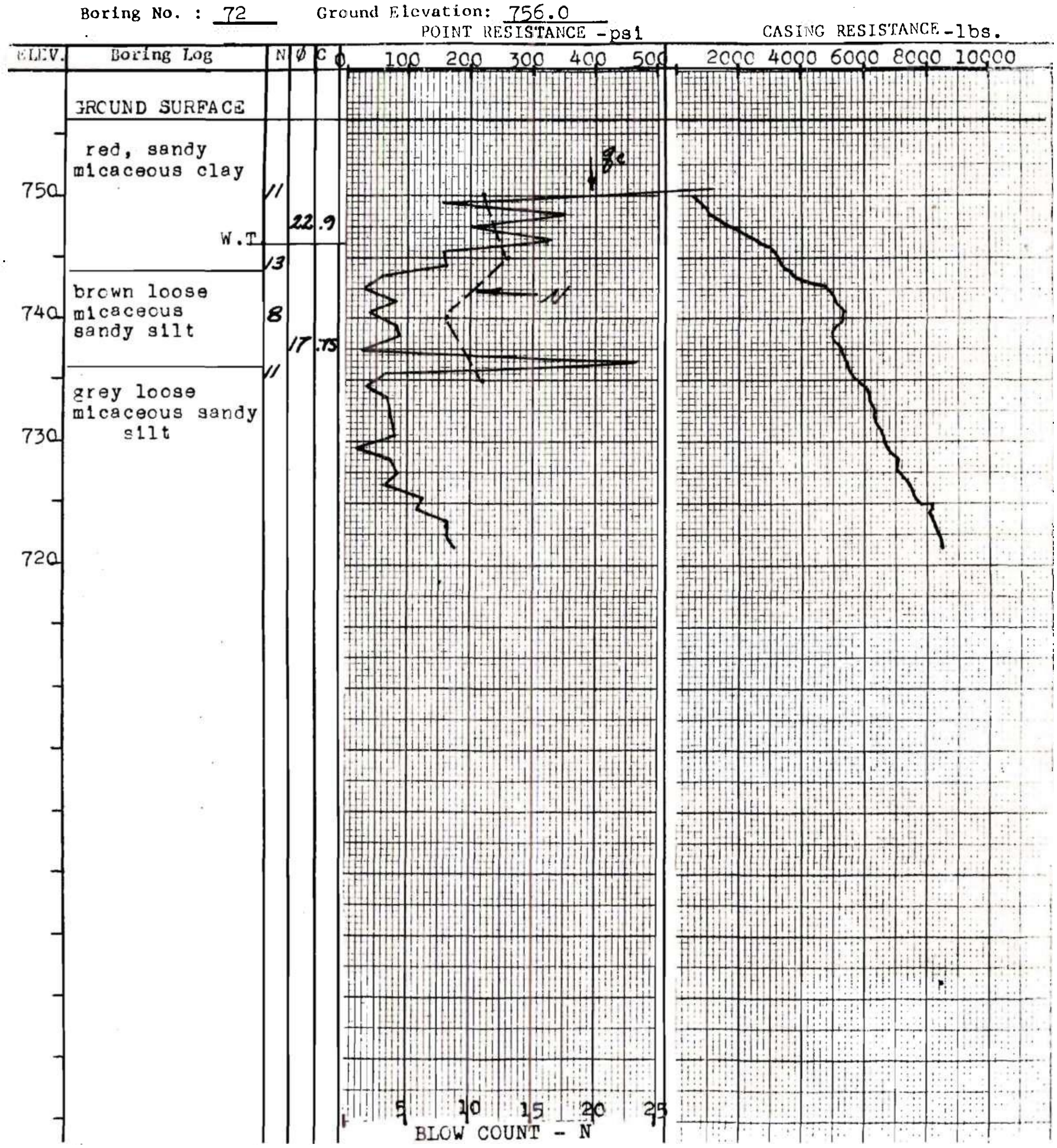
Project No. & County : I-85-2(5) Banks

Party Chief : Crowther

Date : 8/28/62

Bridge Site & Station No. : Site # 2 SR 164

Boring No. : 72 Ground Elevation: 756.0



PENETROMETER LOG

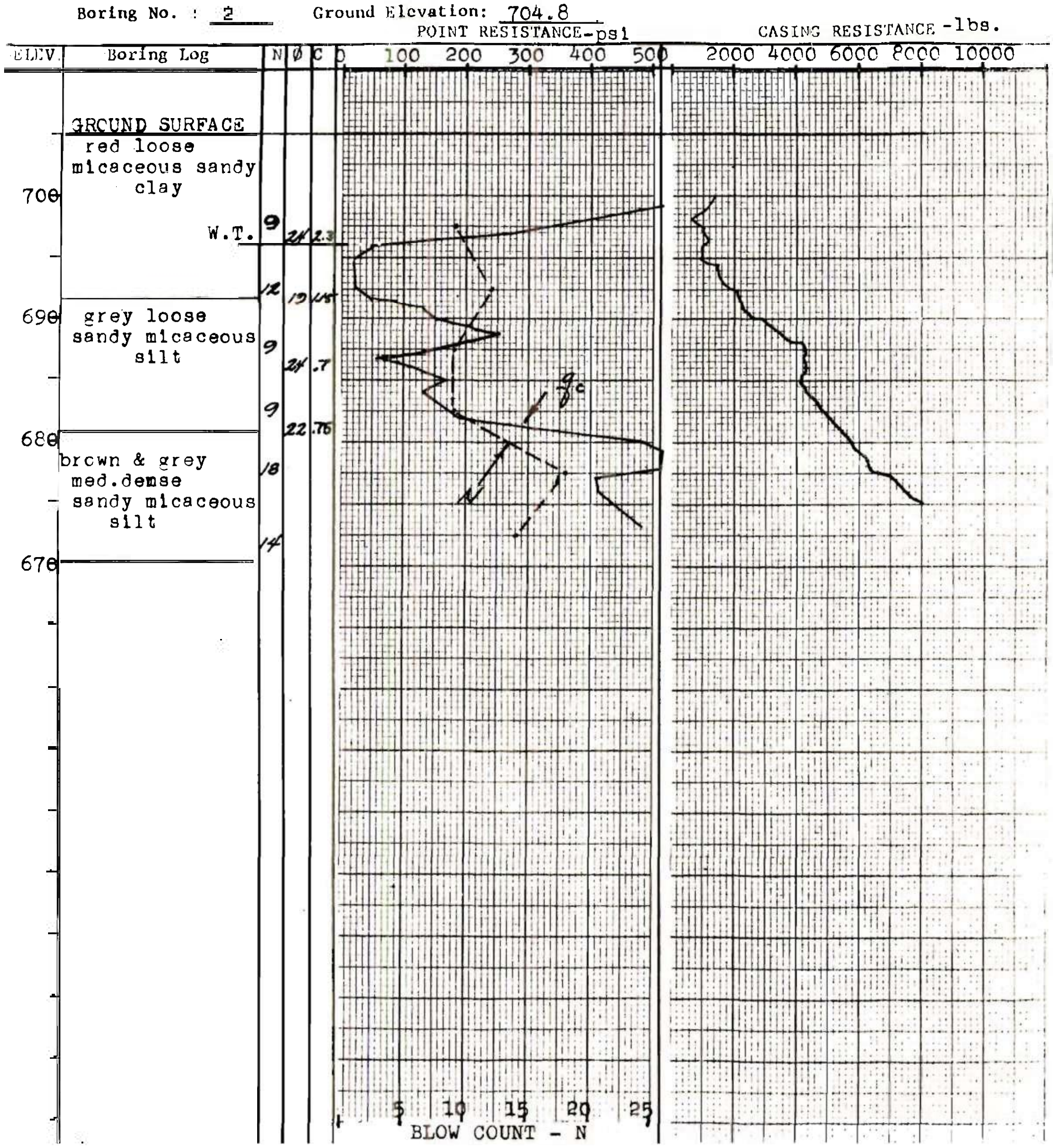
Project No. & County : I-85-2(5) Banks

Party Chief : Crowther

Date : 8/28/62

Bridge Site & Station No. : Site #3 SR 15

Boring No. : 2 Ground Elevation: 704.8



PENETROMETER LOG

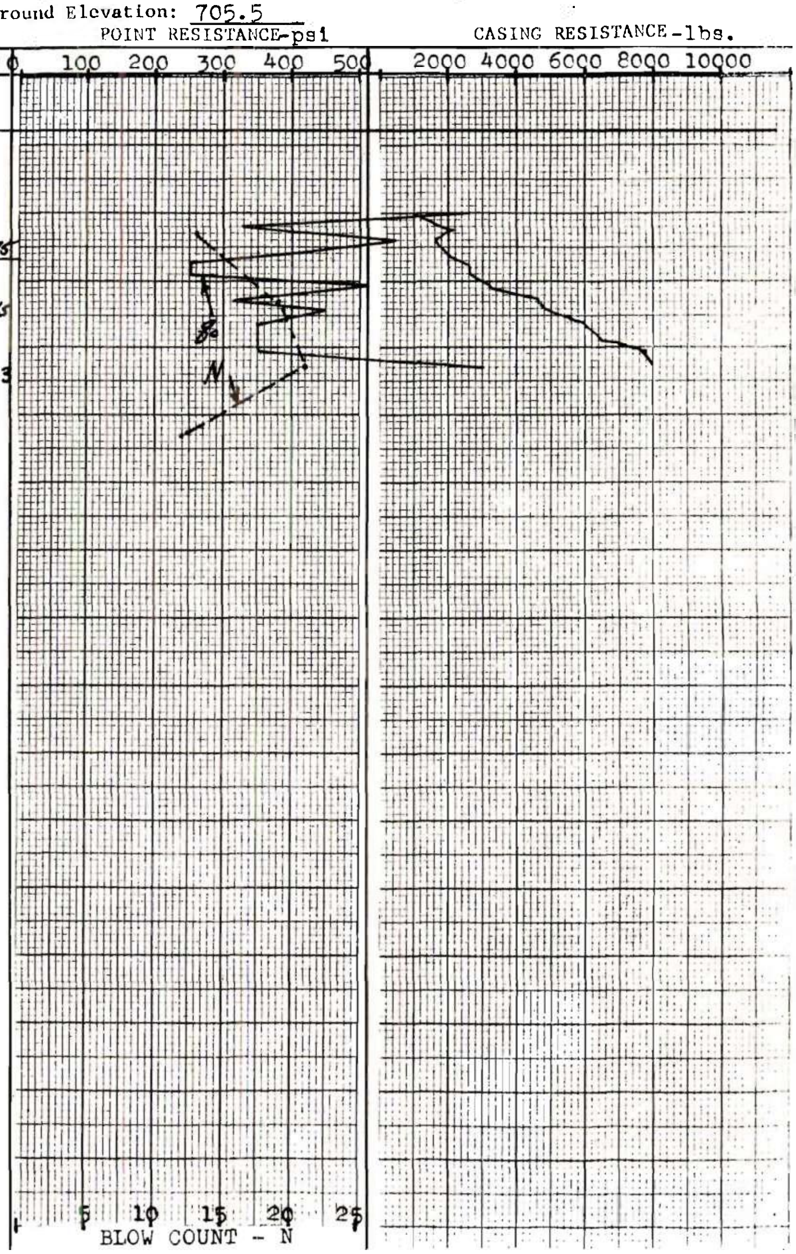
Project No. & County : I-85-2(5) Banks

Party Chief : Crowther
Date : 8/29/62

Bridge Site & Station No. : site #3 SR 15

Boring No. : 4 Ground Elevation: 705.5

ELEV	Boring Log	N	Ø	C
	GROUND SURFACE			
700	red loose micaceous sandy clay			
	W.T.	13	22	.75
	grey, med.dense micaceous silty sand	19	27	1.15
690		21	25	2.3
	grey loose micaceous sandy silt	12		
680				
670				



PENETROMETER LOG

Project No. & County : I-85-2(5) Banks

Party Chief : Growther

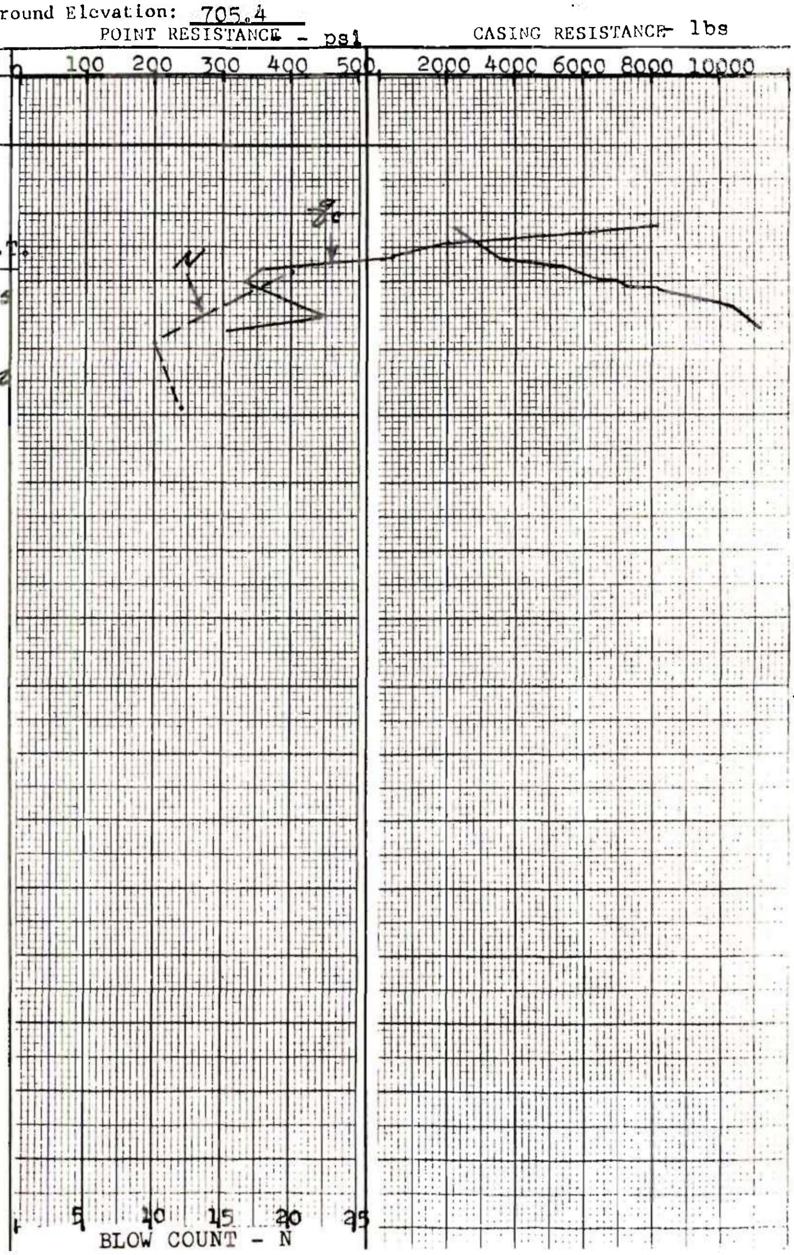
Date : 8/30/52

Bridge Site & Station No. : Site #3 SR 15

Boring No. : 6

Ground Elevation: 705.4

ELEV.	Boring Log	N	Ø	C
	GROUND SURFACE			
700	brown med.stiff micaceous sandy clay			
	brown & yellow loose micaceous sandy silt	20	10	.25
690	brown loose micaceous sandy silt	10	13	1.2
		12		
680				
670				



PENETROMETER LOG

Project No. & County : Cobb Co. Court House

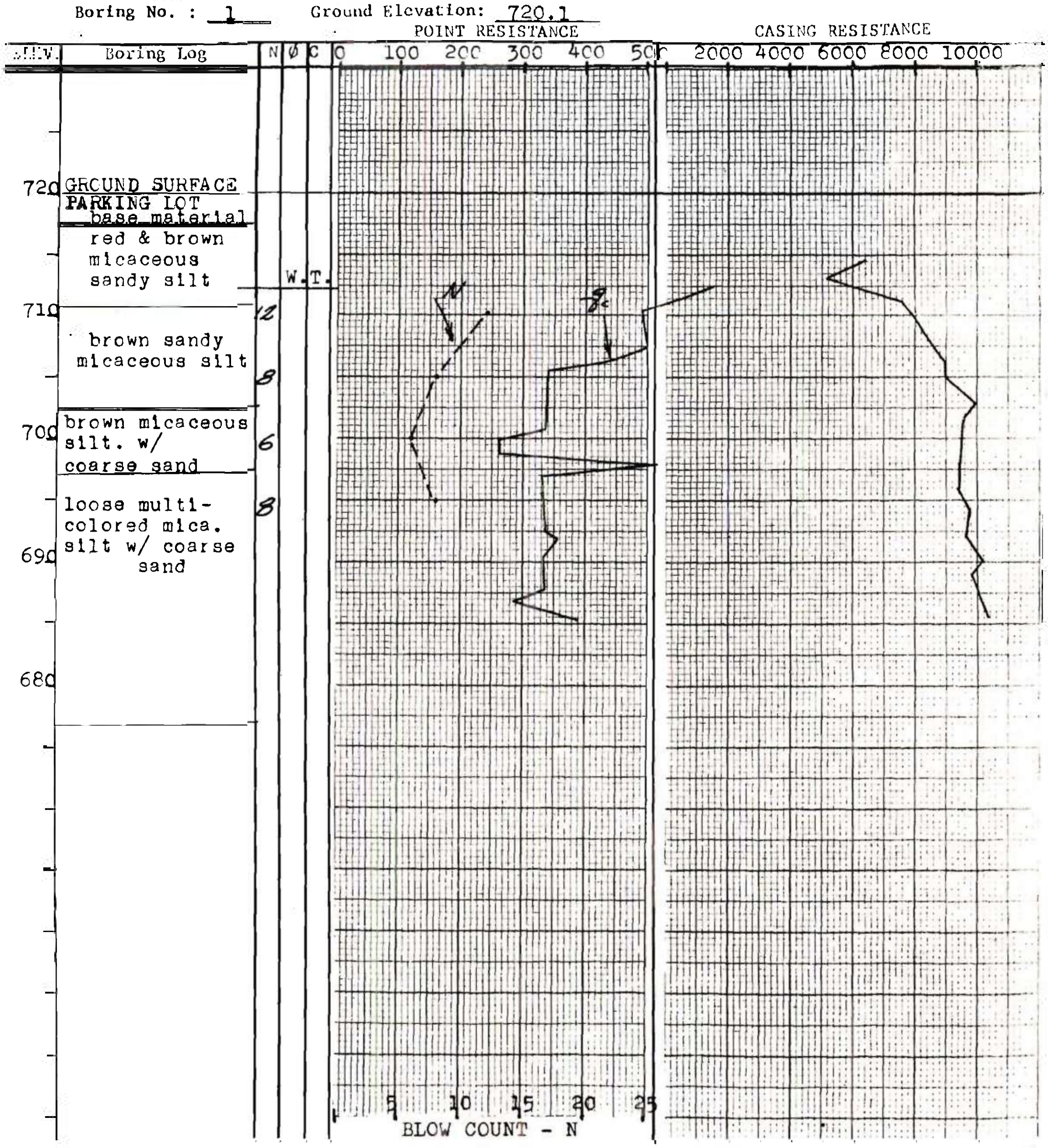
Party Chief : Crowther

Date : 6/14/62

Bridge Site & Station No. : Cobb County Court House (proposed) Site#4

Boring No. : 1

Ground Elevation: 720.1



PENETROMETER LOG

Project No. & County : I-85-1(26) Fulton

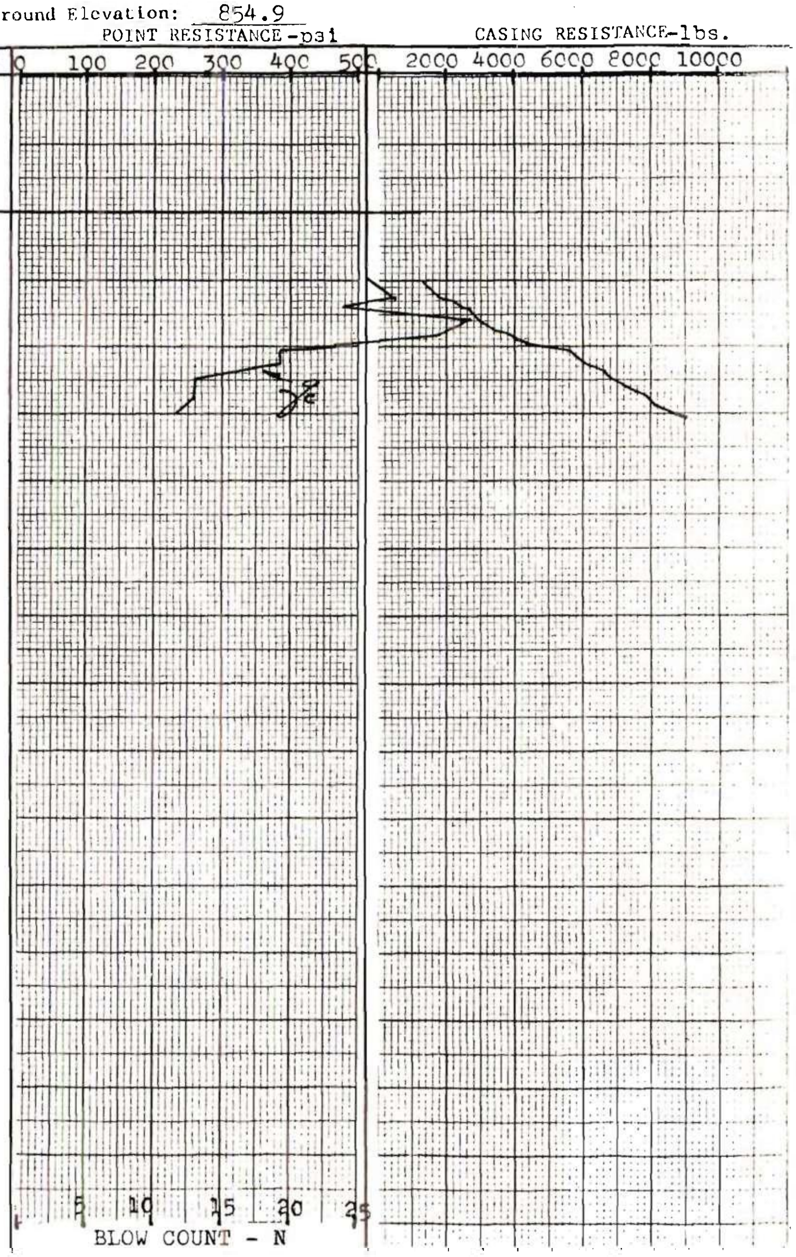
Party Chief : Crowther

Date : 8/20/62

Bridge Site & Station No. : Site # 5 ACL RR Underpass

Boring No. : 5   Ground Elevation: 854.9

ELEV.	Boring Log	N	Ø	C
	GROUND SURFACE			
	tan sandy clay			
850	multi-colored sandy micaceous silt			
840				



PENETROMETER LOG

Project No. & County : I-85-1(26) Fulton

Party Chief : Crowther

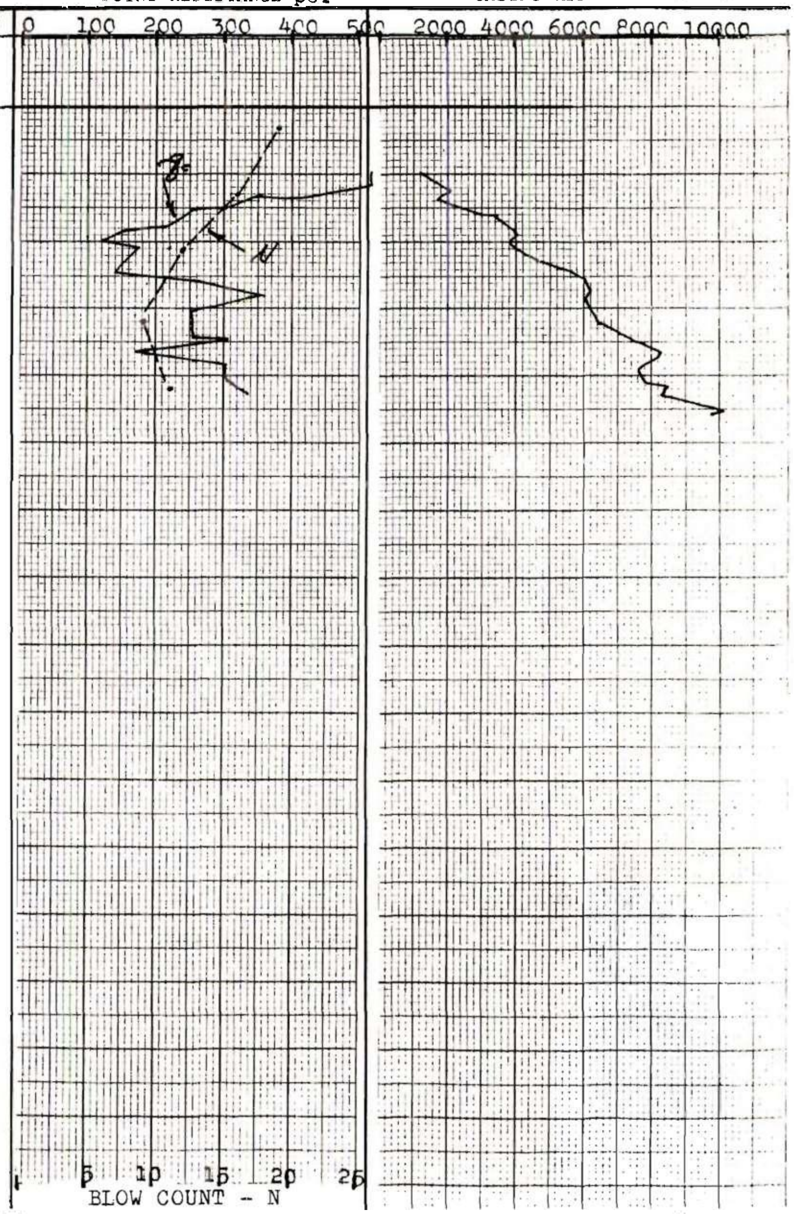
Date : 8/21/62

Bridge Site & Station No. : Site # 5 ACL RR Underpass

Boring No. : 8

Ground Elevation: 855.0

ELEV.	Boring Log	N	Ø	C
	GROUND SURFACE			
	red sandy clay	19		
850	red silty clay	16		
	pink silty micaceous clay	12		
840	multi-colored clayey micaceous silt	5		
		4		
830				



PENETROMETER LOG

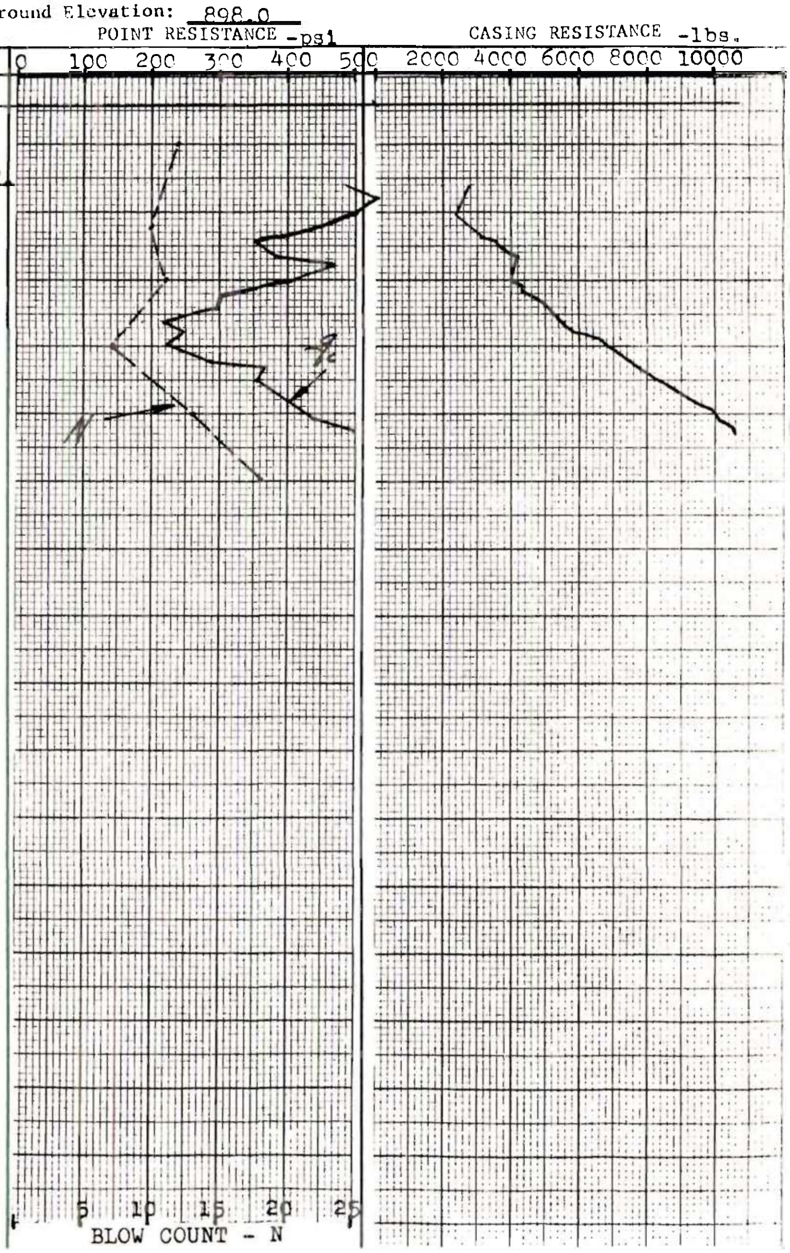
Project No. & County : F-035 Rockdale

Party Chief : Crowther
Date : 8/31/62

Bridge Site & Station No. : Site #6 Ga.RR Overpass

Boring No. : 1   Ground Elevation: 898.0

ELEV.	Boring Log	N	Ø	C
	GROUND SURFACE			
	yellow sandy silty, micaceous clay	12		
			W.T.	
890	yellow and white loose sandy micaceous silt	10		
		11		
880		7		
		13		
870	red.dense sandy micaceous silt	18		
860				
850				



PENETROMETER LOG

Project No. & County : F-035 Rockdale

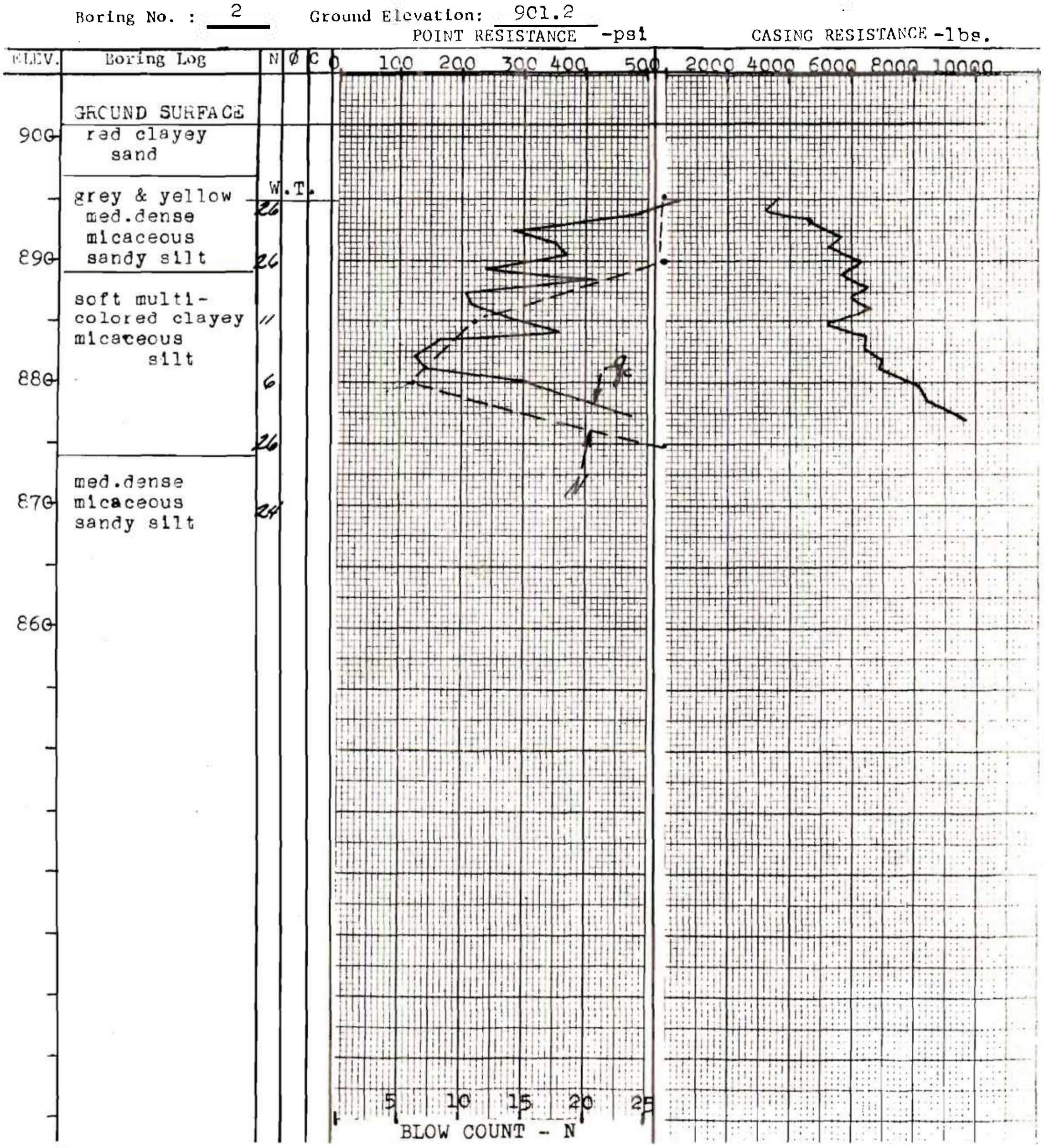
Party Chief : Crowther

Date : 8/31/62

Bridge Site & Station No. : Site # 6 Ga. RR Overpass

Boring No. : 2

Ground Elevation: 901.2



UNDISTURBED SAMPLE PROPERTIES

AND

MOHR'S DIAGRAMS

UNDISTURBED SAMPLE PROPERTIES

Project BA(5) SP-1645 Clarke

Sample No.	Wet Density	w%	G_s	e	ϕ^o	C-KSF
5-1U	115.8	33.3	2.63	.876	-	-
5-2U	121.6	28.8	2.69	.774	-	-
5-4U	123.7	25.3	2.60	.658	-	-
6-1U	107.7	22.3	2.65	.591	10	.9
6-2U	119.2	26.5	2.63	.696	5	.6
6-3U	114.8	33.8	2.61	.881	31	.3
6-4U	122.1	23.3	2.68	.624	15	1.1

Project F-035 Rockdale

Sample No.	Wet Density	w%	G_s	e	ϕ^o	C-KSF
1-1U	101.7	19.4	2.55	-	23.5	1.25
1-2U	108.7	24.6	2.57	.632	24.5	.85
1-3U	115.8	36.0	2.54	.914	19.0	.95
1-4U	114.0	37.9	2.56	.970	20	1.0
2-1U	106.1	16.5	2.58	-	18	1.95
2-2U	112.5	28.2	2.61	.735	29	.90
2-3U	118.6	29.1	2.56	.742	41	00
2-4U	101.1	32.0	2.53	.812	23.5	1.2

Project I-85-1(26) Fulton - ACL RR

Sample No.	Wet Density	w%	G_s	e	ϕ^o	C-KSF
7-1U	117.1	29.4	2.64	.718	22	.4
7-2U	118.3	29.9	2.63	.787	14	1.0
7-3U	119.2	31.8	2.60	.827	19	.6
7-4U	121.9	24.6	2.63	.647	-	-

UNDISTURBED SAMPLE PROPERTIES

Project I-85-2(5) Banks

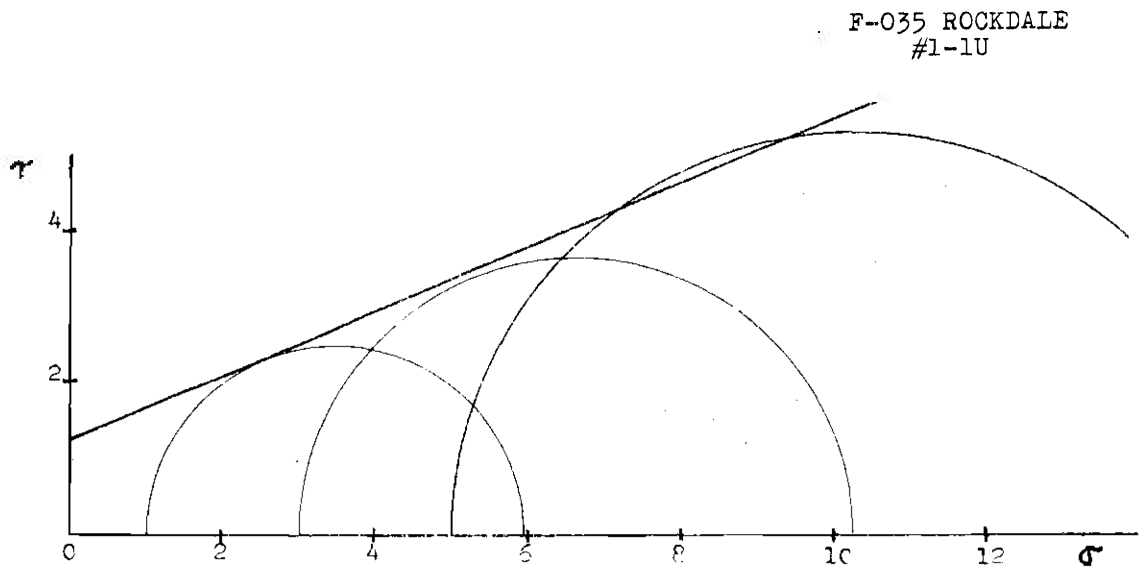
SR-15

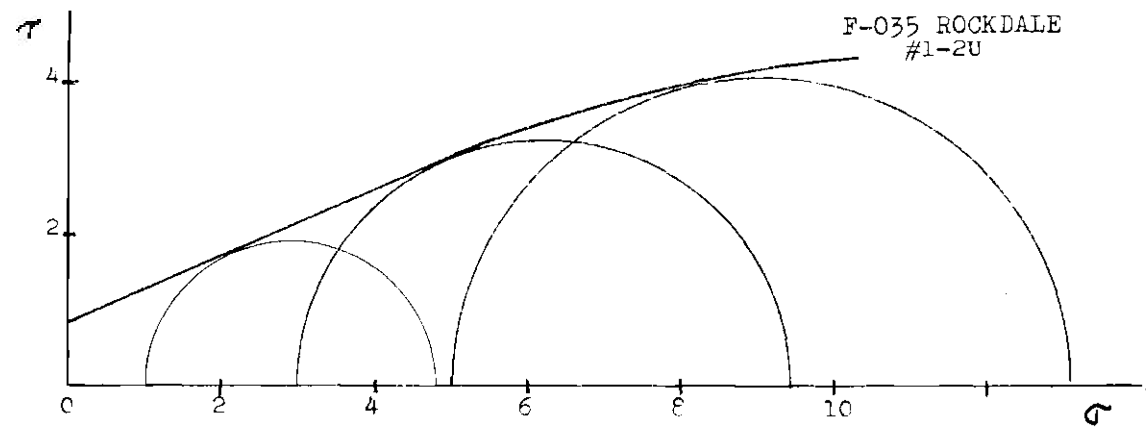
Sample No.	Wet Density	w%	G_s	e	ϕ^o	C-KSF
2-1U	111.9	31.2	2.65	-	24	2.3
2-2U	96.4	45.7	2.73	1.271	19	1.15
2-3U	101.6	38.4	2.60	.995	24	.7
2-4U	118.2	31.1	2.65	.825	22	.75
4-1U	116.5	22.7	2.58	-	27	1.15
4-2U	118.6	35.0	2.67	.935	23	2.3
4-3U	114.0	37.5	2.59	.971	-	-
4-4U	120.2	26.2	2.63	.687	15	1.35
6-1U	115.9	21.1	2.65	-	-	-
6-2U	109.2	28.2	2.59	.730	-	-

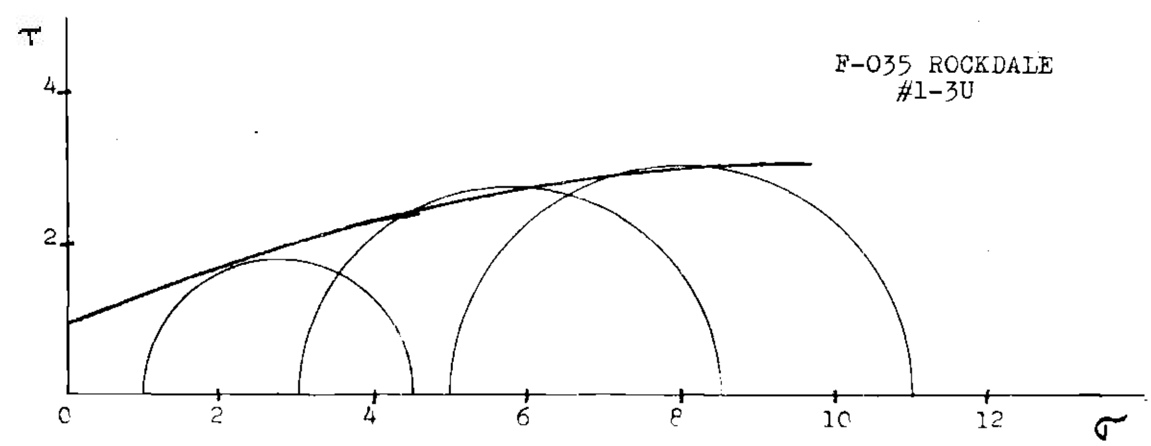
Project I-85-2(5) Banks

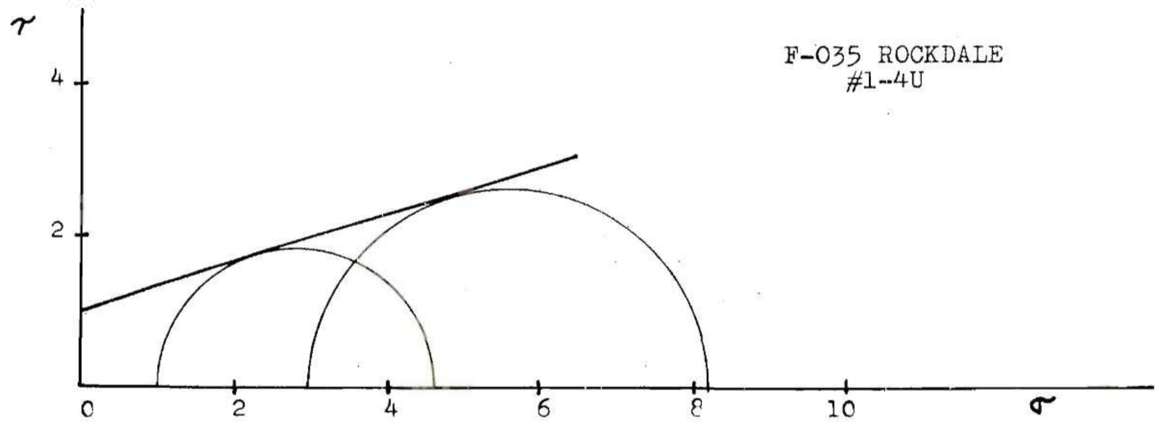
SR-164

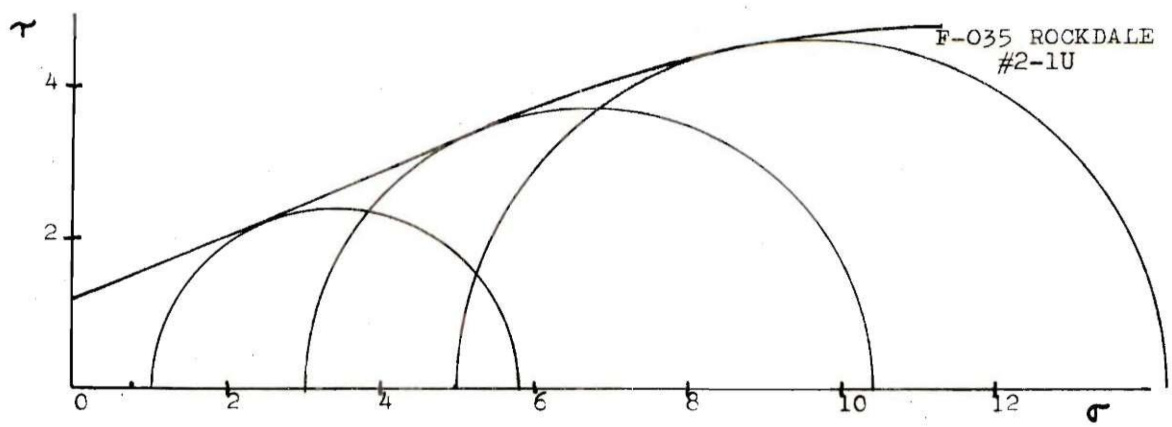
Sample No.	Wet Density	w%	G_s	e	ϕ^o	C-KSF
70-1U	134.7	10.9	2.59	-	-	-
70-2U	112.4	32.9	2.59	.852	-	-
70-3U	111.3	27.0	2.57	.694	14	2.35
70-4U	115.8	37.9	2.65	1.01	21	.45
71-1U	123.0	12.7	2.60	-	15	1.7
71-2U	121.5	39.0	2.68	1.015	21	.35
71-3U	111.2	42.1	2.64	1.11	18	.35
71-4U	116.8	33.2	2.66	.884	14	.25
72-1U	127.2	19.1	2.60	-	22	.9
72-2U	111.7	35.8	2.60	.930	-	-
72-3U	122.8	26.2	2.62	.685	17	.75
72-4U	112.9	38.6	2.65	1.025	-	-

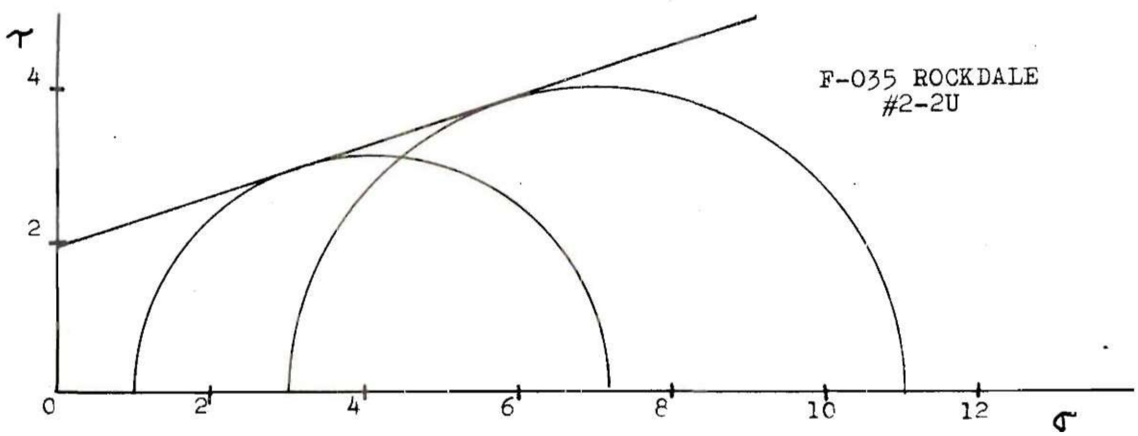

F-035 ROCKDALE
#1-1U
τ
4
2
0
2
4
6
8
10
12
σ

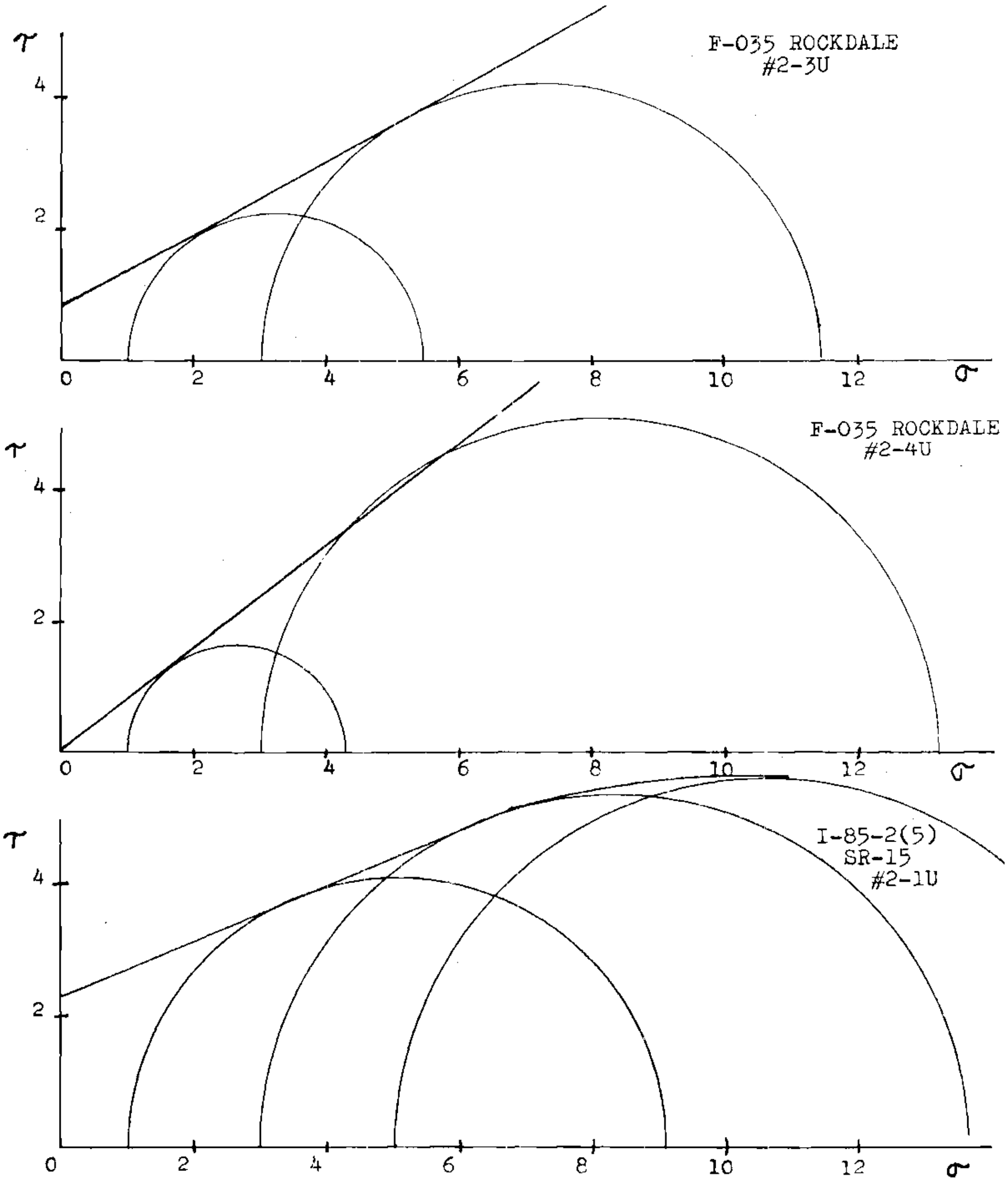

F-035 ROCKDALE
#1-2U
τ
4
2
0
2
4
6
8
10
σ

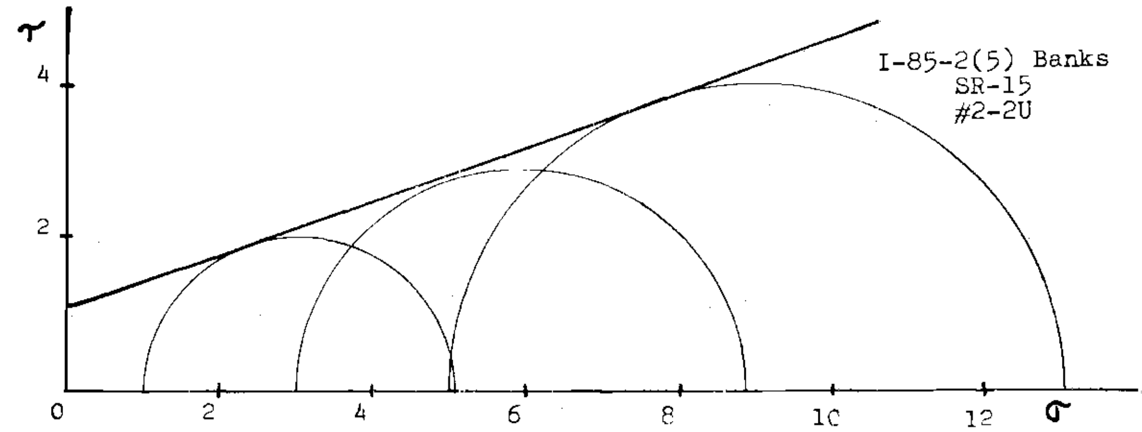

F-035 ROCKDALE
#1-3U
τ
4
2
0
2
4
6
8
10
12
σ

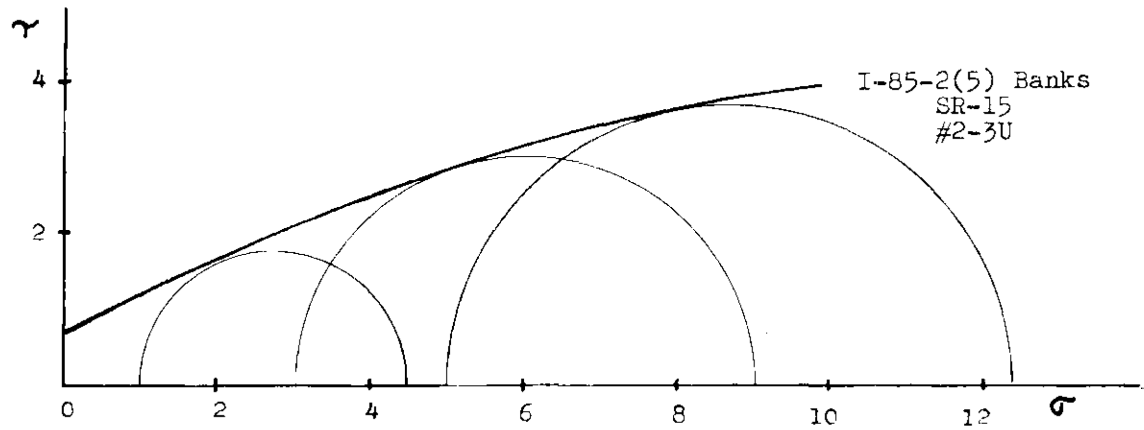

F-035 ROCKDALE
#1-4U
τ
4
2
0
2
4
6
8
10
σ

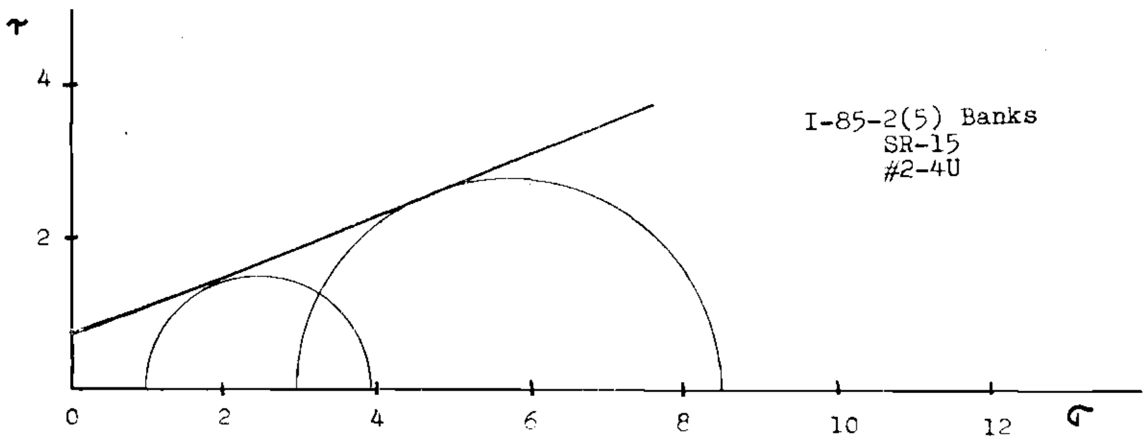

F-035 ROCKDALE
#2-1U
τ
4
2
0
2
4
6
8
10
12
σ

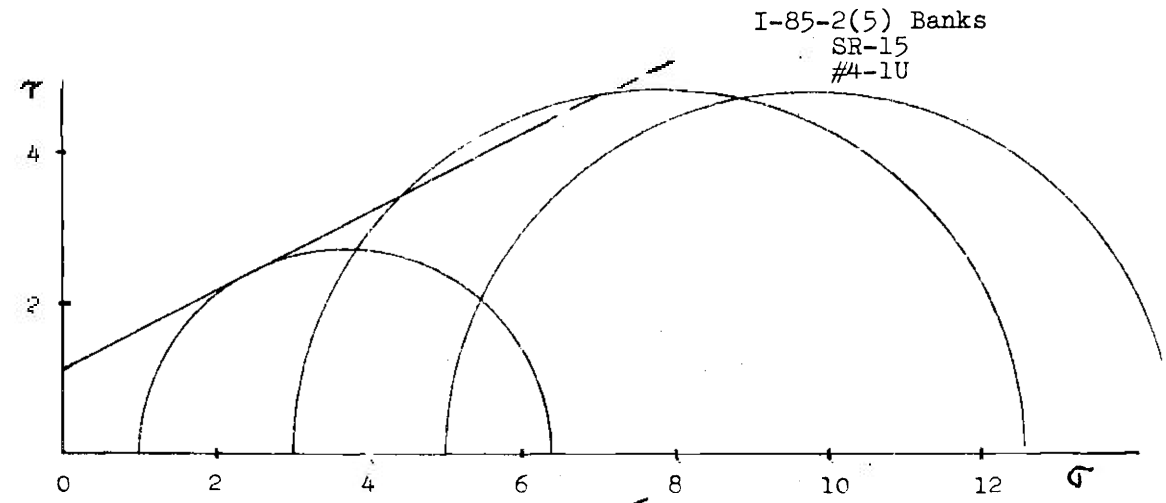

F-035 ROCKDALE
#2-2U
τ
4
2
0
2
4
6
8
10
12
σ


F-035 ROCKDALE
#2-3U
F-035 ROCKDALE
#2-4U
I-85-2(5)
SR-15
#2-1U
τ
σ
0
2
4
6
8
10
12


I-85-2(5) Banks
SR-15
#2-2U
τ
4
2
0
2
4
6
8
10
12
σ


I-85-2(5) Banks
SR-15
#2-3U
τ
4
2
0
2
4
6
8
10
12
σ


I-85-2(5) Banks
SR-15
#2-4U
τ
4
2
0
2
4
6
8
10
12
σ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SR-15
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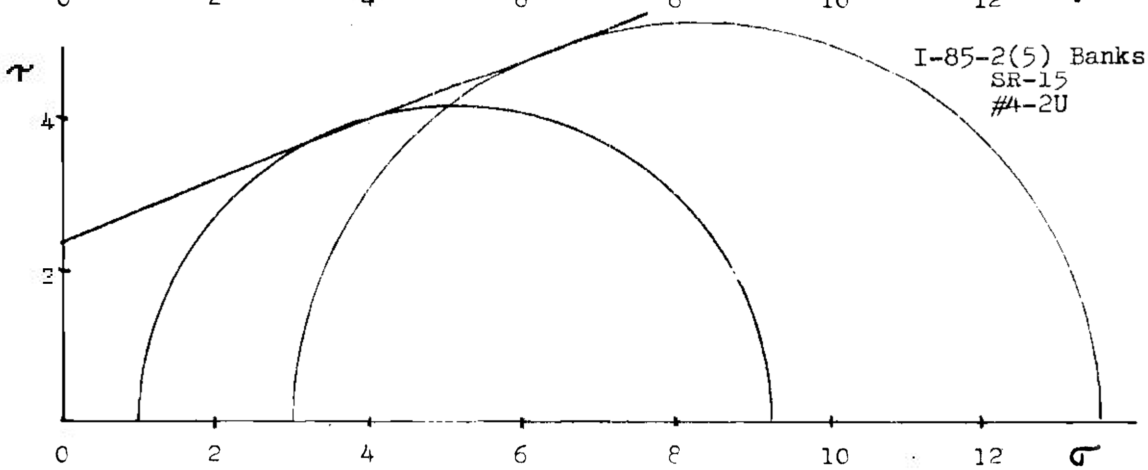
σ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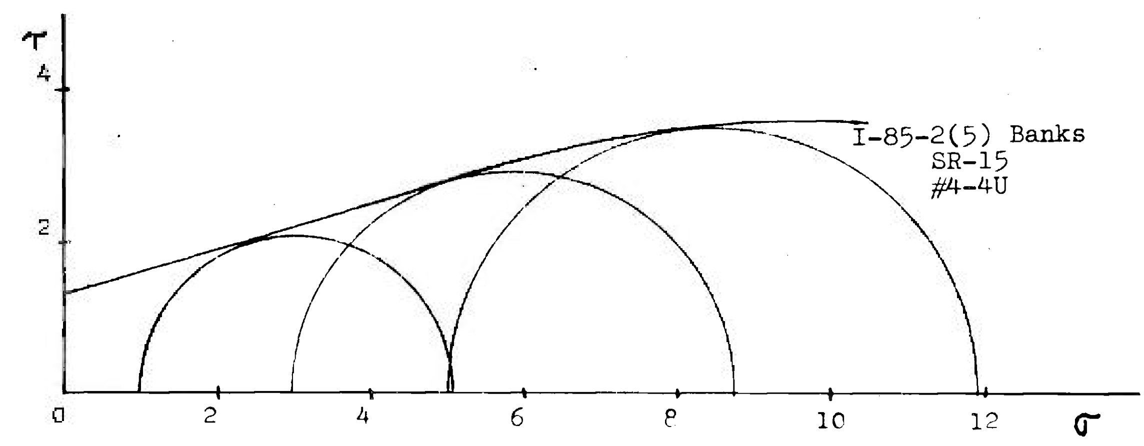
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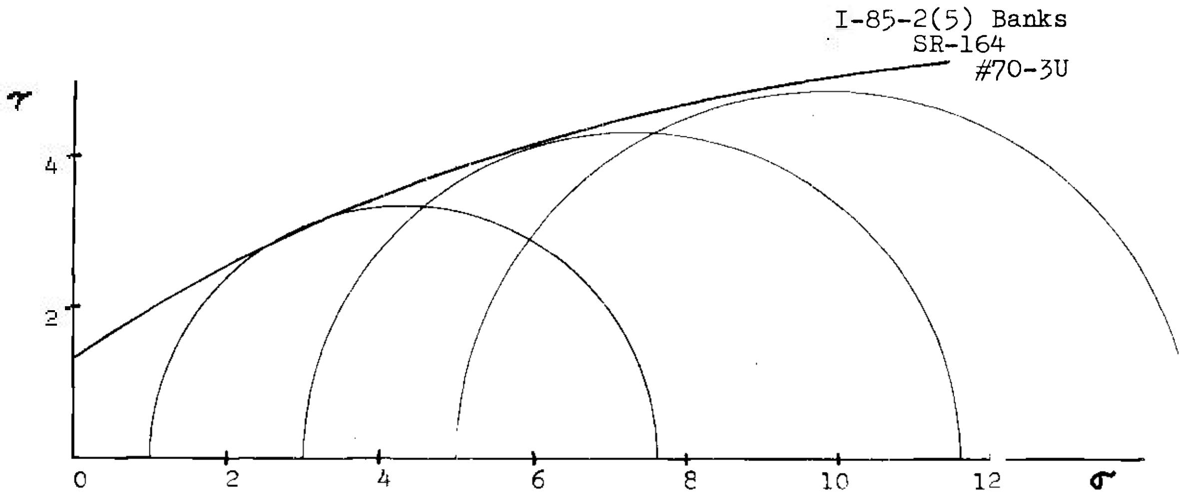
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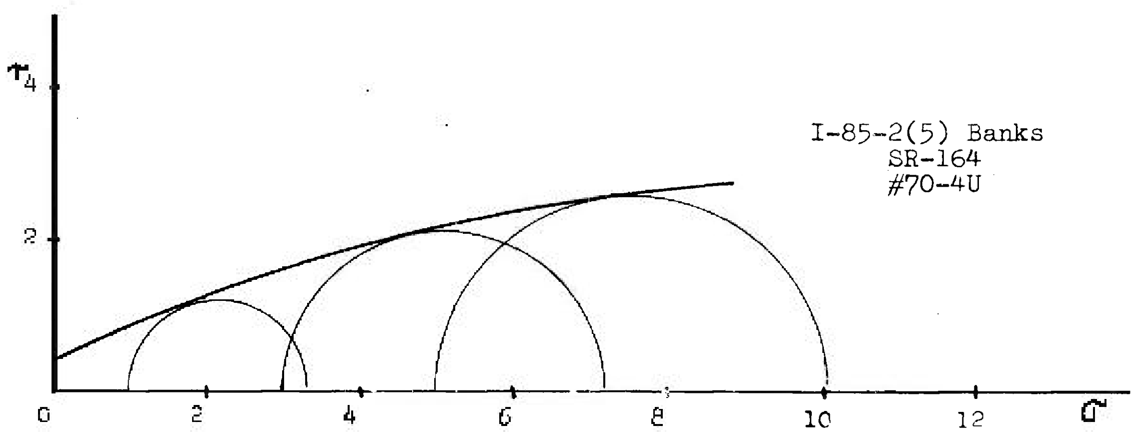
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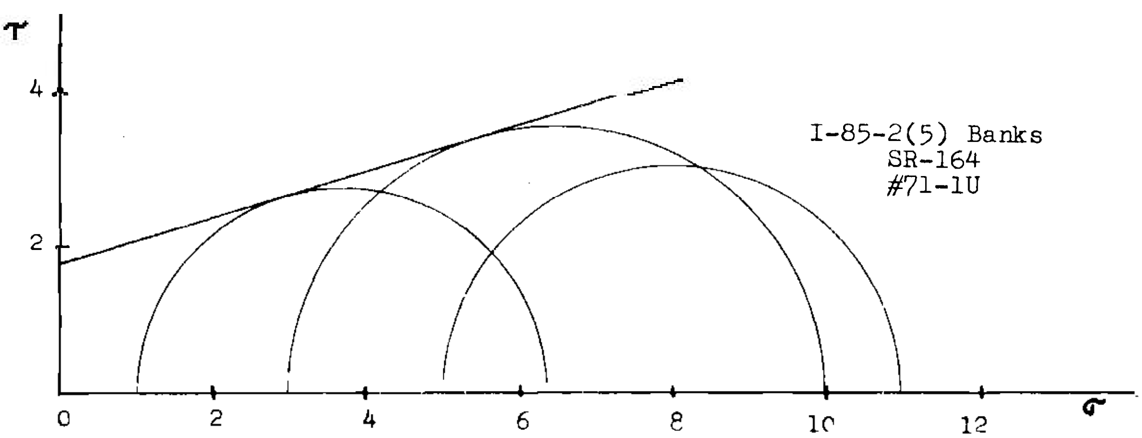
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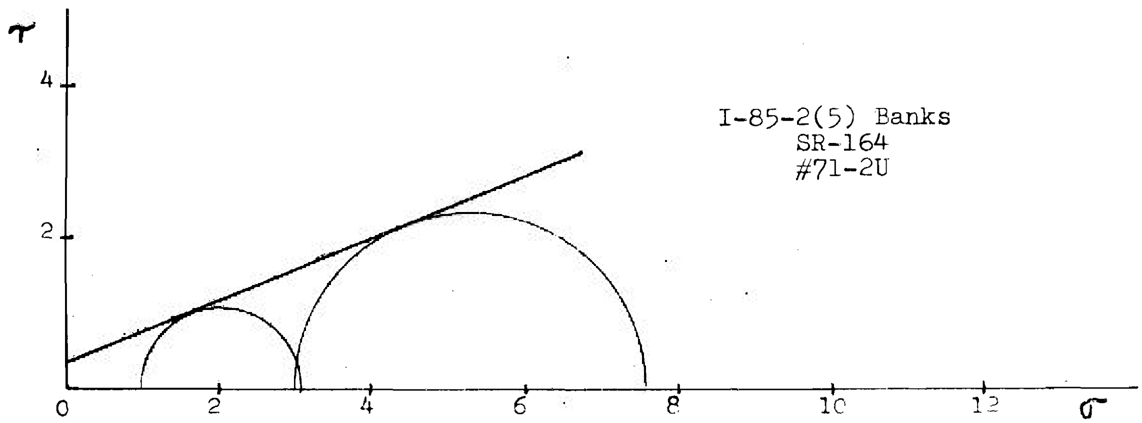
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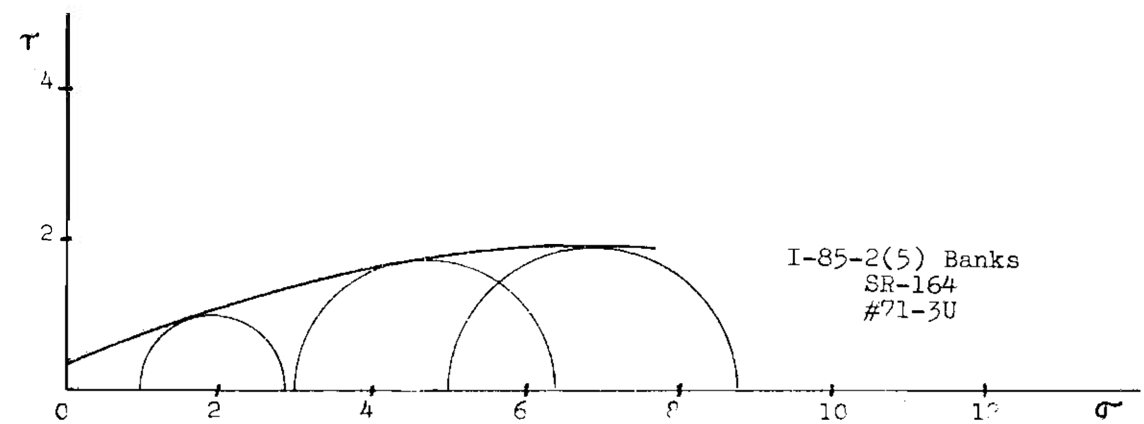
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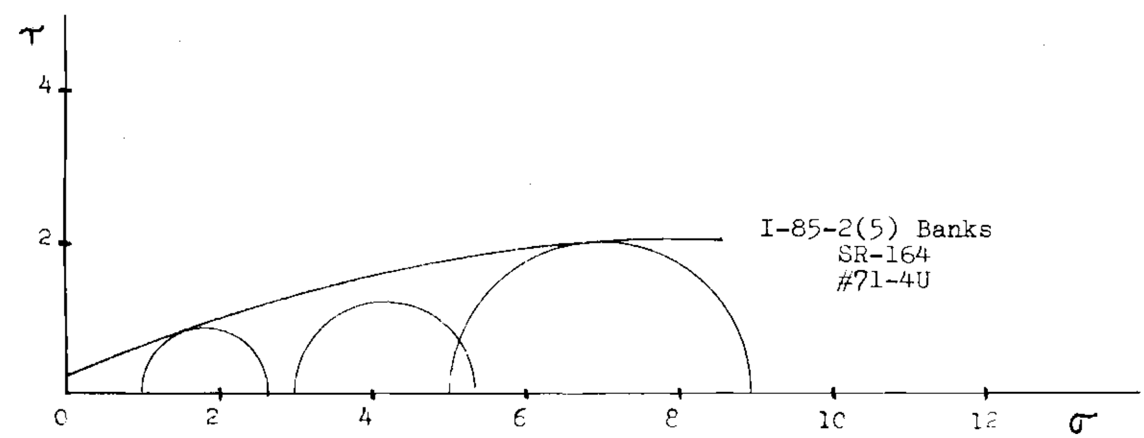
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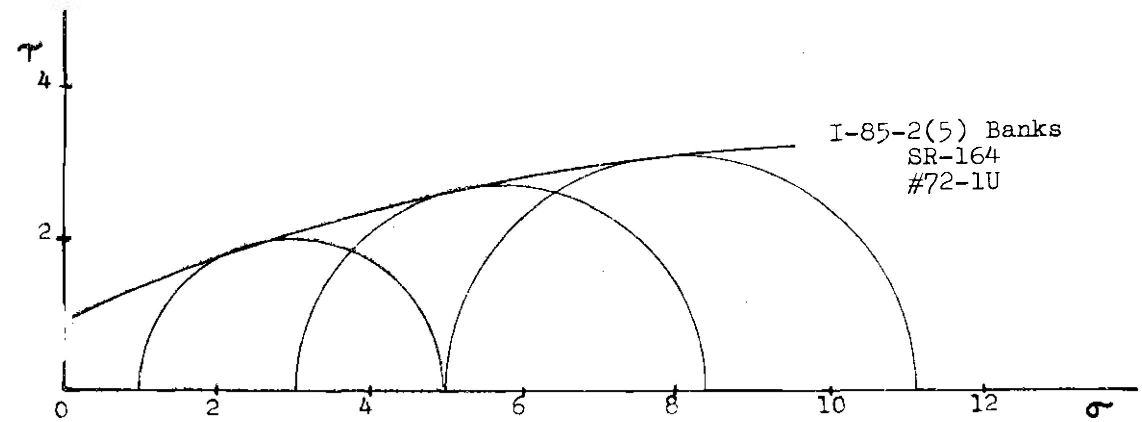

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SR-164
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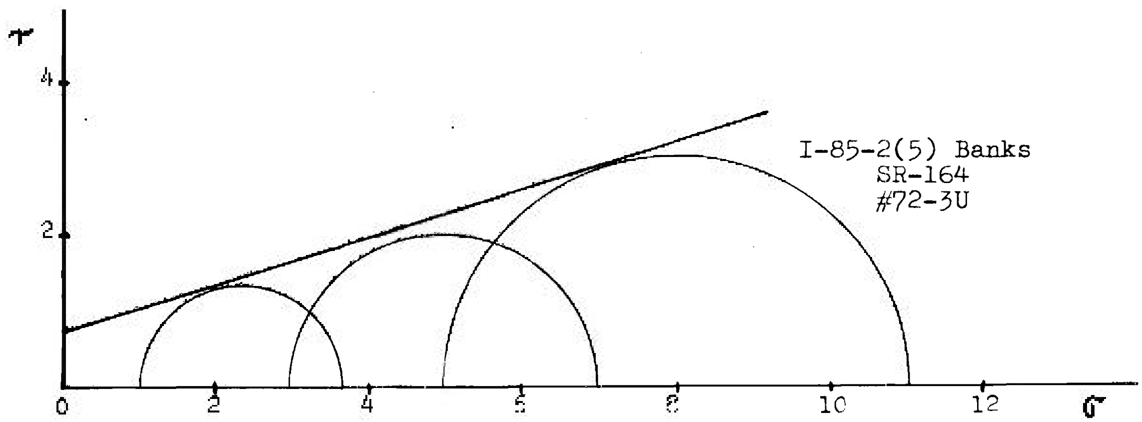

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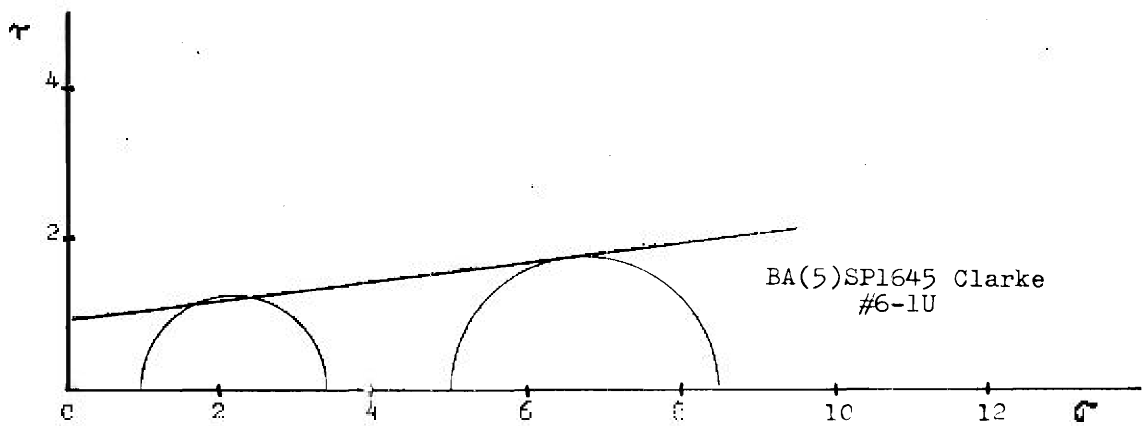

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12
σ
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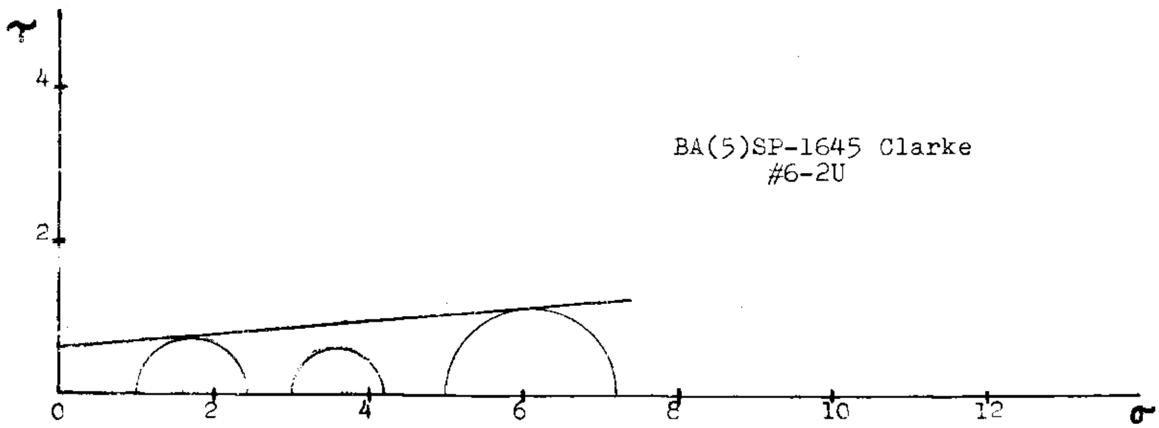
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τ
σ
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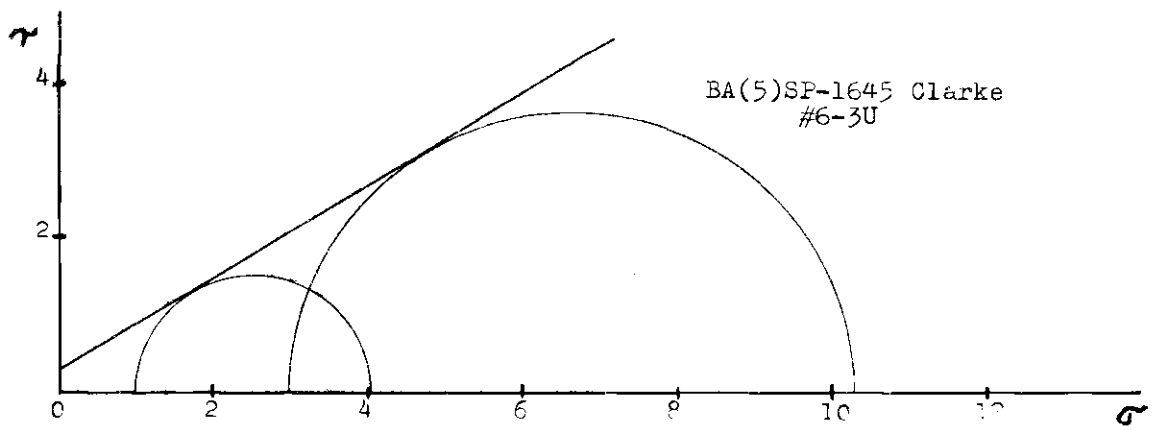
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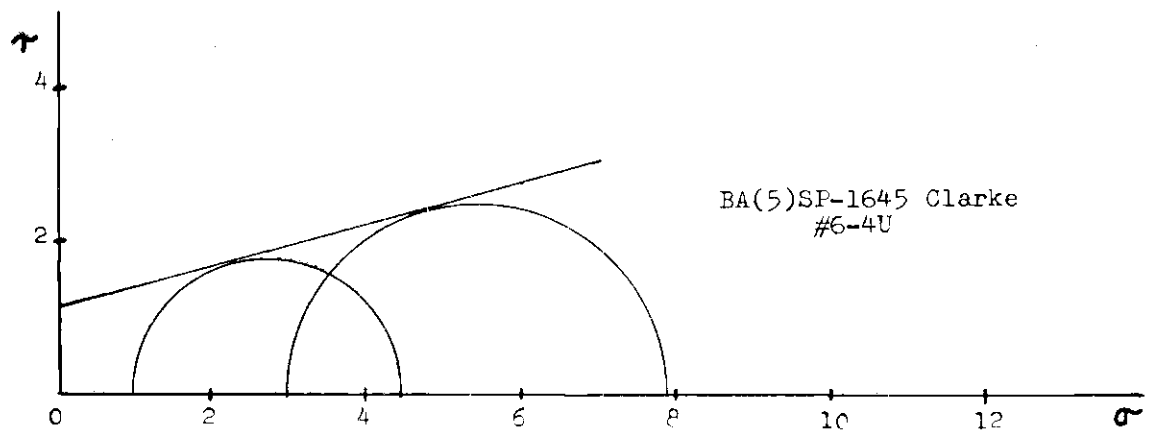
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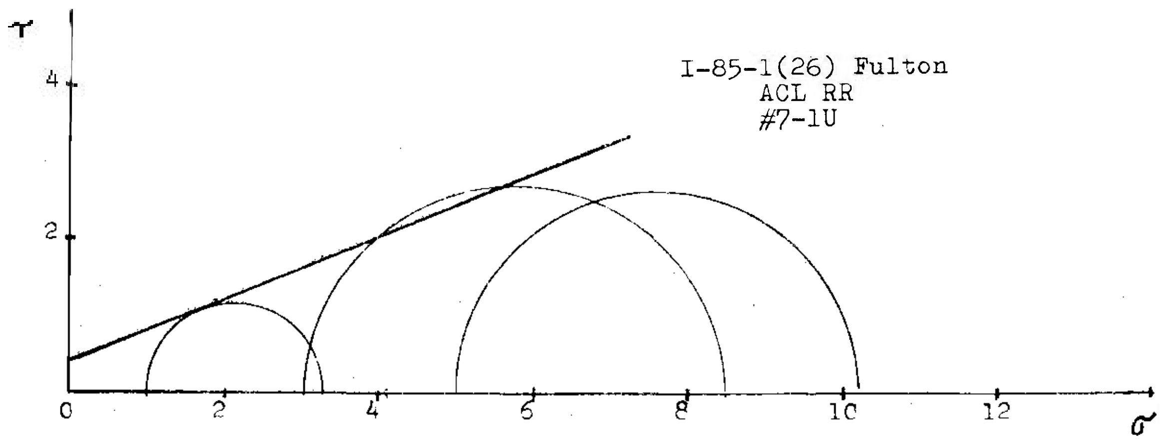
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ACL RR
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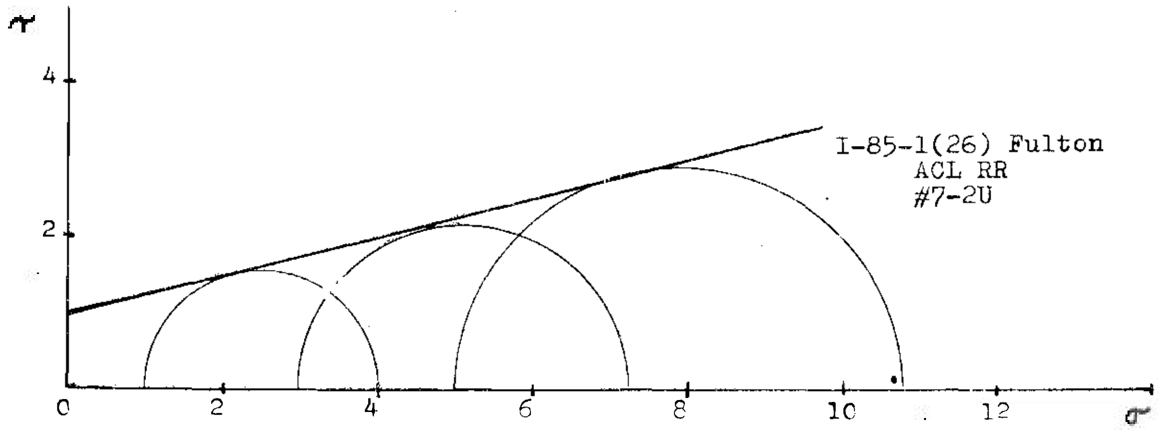
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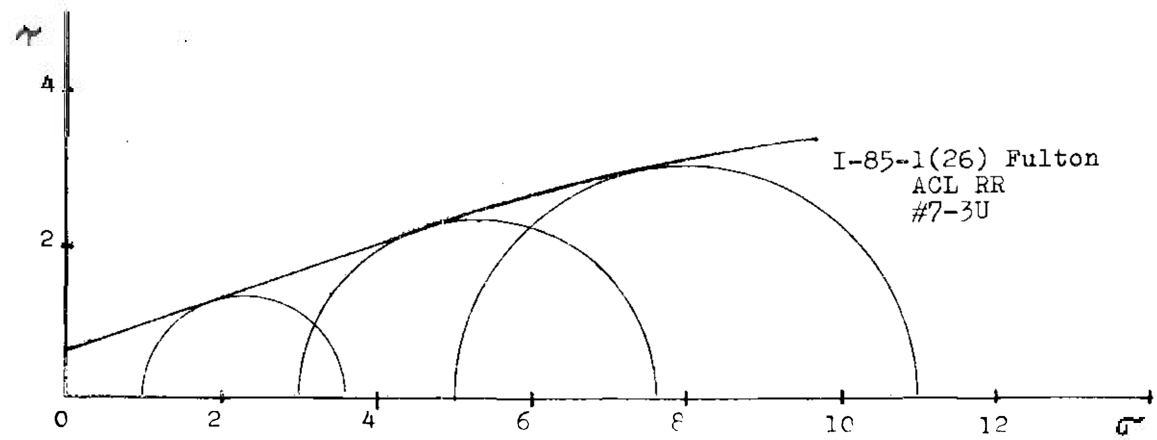
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